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Permeability of stabilized clay

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ABSTRACT OF THE FINAL WORK

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Clays in the Nordic Countries are mainly very soft and sensitive due to their post-glacial origin. These clays have behaviours and principal parameters which differ from most clays reported in the literature. But in contrast to the negative properties, Nordic clays have very low permeability and good resistance to harmful chemicals.

Contaminants that have infiltrated into the ground tend to be diffused into the surroundings along with the seepage of water. The spreading of pollutants can be prevented by reducing water exchange between a contaminated area and its environment. Because Finnish clay has very low permeability, it is potentially a good material for barriers in landfill and disposal sites.

In this way, in the beginning of the year 2006, a project aiming at improvement of sensitivity of clays was started at the Soil Mechanics and Foundation Engineering Laboratory of the Helsinki University of Technology (HUT). This project tries to improve clays softness by stabilizing with cement. One of the sub-tasks of the project concerns permeability.

This report concerns permeability of the soft clays in order to collaborate on the project mentioned before. So, at the beginning of the year 2006 some samples were tested for this report with the objective to study their behaviours, to test different methods and models and to compare them. The samples tested are from Vanttila, Otaniemi and Murro.

All the samples were tested in the laboratory by using a conventional oedometer. In this study permeability was measured as described by Leroueil (1990) in the conventional oedometer by applying a water pressure at the bottom of the sample after each loading step and measuring the pressure heads at the beginning and at the end of the test. Also, the permeability was obtained by means of indirect methods such as Taylor and Casagrande as well as of others parameters to have a greater knowledge about the soft clays. Permeability and coefficient of consolidation have been the main parameters analyzed in this study.

This report analyzes methods and models developed by the Soil Mechanics and Foundation Engineering Laboratory about permeability and coefficients of consolidation compared with other methods and models. The aim is to compare the differences and similarities among them and declare the possible advantages of the models or methods proposed for Soil Mechanics and Foundation Engineering Laboratory.

The analysis of the results and the comparisons made verifies that the permeability and the coefficient of consolidation models proposed by Soil Mechanics and Foundation Engineering Laboratory give at least results similar to other models and they are more reliable because of the fact that more precise models have been used (i.e. strain-stress relationship of Janbu).

It is also a very important task in this report to describe the way in which the Soil Mechanics and Foundation Engineering Laboratory has developed the Unconfined Compression Tests and the Permeability Tests since the preparation of the stabilized sample until the obtaining of results.

PREFACE

The aim of this report is to study the permeability of the cement stabilized clays of Finland in order to collaborate with a three years project (2006-2008) carried out by the Soil Mechanics and Foundation Laboratory of the Helsinki University of Technology (HUT).

Several Unconfined Compression Tests and Permeability Tests have been developed from February to June 2006 at the Soil Mechanics and Foundation Laboratory, but, because it is very soon to achieve conclusions from these, see the big project started at the beginning of the year, two objectives will be met in the course of this writing.

The first of them is the practical one and it has offered to me the opportunity to embody the laboratory work I have done during my time at the HUT. It deals with describing accurately the steps that shall be followed to carry out the Unconfined Compression Tests and the Permeability Tests at the Soil Mechanics and Foundation Laboratory of the Helsinki University of Technology (HUT) since the preparation and preservation of the samples until the presentation of results.

The second objective is the theoretic one and it has dealt with reading the most important bibliography in the field of this Final Thesis and gathering the most interesting aspects on these papers. Not only permeability has been treated, settlements in stabilized clays what is one of the most important pillars to explain the permeability of soils, has been also dealt.

The realization of this report was offered to me by the Soil Mechanics and Foundation Engineering Laboratory of the Helsinki University of Technology (HUT) through Professor Olli Ravaska in order to finalize my final thesis, what has supplied me great benefit both formatively and personally. I would like to thank Olli Ravaska for offering me this possibility and also for invaluable helping hand of both him and Matti Lojander. I would like to thank as well all the people who have helped me and have made that my stay at this Laboratory has been so pleasant: Irmeli for the bureaucracy; Jarmo, Matti and Timo for the explanations at the laboratory; Hassan for

the laboratory results; Jonni at the computers; Henkku for so many things, and Elise for the advices and her friendship.

Helsinki, November 2006

LIST OF SYMBOLS

The symbols used in this text are presented below for better compression:

a	cross-sectional area of the standpipe, acceleration
A	Constant at HUT's model; cross section of the sample
B	constant at m definition; constant at HUT model
c	cohesion
C	constant at Taylor's method
C_c	compression index
C_u	undrained shear strength
c_v	coefficient of consolidation
d	length of the maximum drainage path
D	diameter of vane at the vane test
e	void ratio
e_0	void ratio corresponding to any convenient value of the effective stress σ'_0 ; reference void ratio; initial void ratio
g	acceleration due to gravity
E'_{oed}	secant modulus of the oedometer stress-strain curve
h	total head loss
H	height of vane at the vane test
h_1	initial head at t_1
h_2	final head at t_2
i	hydraulic gradient
I	constant at Raymond's method
k	coefficient of permeability
K	constant at the fall cone test formulation

k_0	reference permeability and e_0 the initial void ratio
k_0	permeability at zero strain
k_I	constant at m definition
k_{10}	coefficient of permeability at 10°C
k_c	vertical permeability corresponding to an arbitrary stress at Fox and Baxter's method
k_n	apparent value of k at zero void ratio e_0
k_T	coefficient of permeability at ambient temperature (m/s)
L	height of the sample
LL	liquid limit
m	model parameter, Janbu; mass at fall cone test
M	deformation modulus
P	force vector at fall cone test
PI	plasticity index
PL	plastic limit
q	rate of flow
Q	vertical weight of the cone at the fall cone test
s	coefficient of sensibility
S_{r0}	initial saturation at Kulklik's method
T	torque at failure (vane test); water temperature
t_{50}, t_{90}	time for 50% and 90% of the consolidation respectively
T_v	time factor
u	pore water pressure
U_v	proportion of the pore pressure dissipated after time t and also the proportion of the total volume change which has taken place at that time
\bar{U}	the mean value of U_v
v	apparent velocity of flow

w	water content
z	depth of penetration at fall cone test
α	Constant; HUT permeability model; correlation factor
β	model parameters Janbu; angle of the cone at the fall cone test
γ_w	unit weight of water.
ε_1	deformation of the normal axis
φ	friction angle
σ_c	preconsolidation pressure; arbitrary stress at Fox and Baxter's method
σ_n	normal stress
σ_r	reference stress of 100 kPa at Janbu model
σ_0'	initial effective vertical stress at Janbu model
σ_1	normal tension in the direction in which we apply the load
σ_3	the normal tension in the radial direction of the sample
τ	shear strength
τ_f	shear strength calculated with the fall cone test

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1. INTRODUCTION

1.1 Objective.

This report is a part of a three years (2006 – 2008) research project called “Use of soft clay in protection barriers” founded by Finnish Academy.

Contaminants that have infiltrated into the ground tend to be diffused into the surroundings along with the seepage of water. The spreading of pollutants can be prevented by reducing water exchange between a contaminated area and its environment. Nowadays there are a variety of natural and synthetic materials available that prevent water exchange between a contaminated area and its environment. The most common mineral barriers are compacted clay liners, soil-bentonite mixtures and geosynthetic clay liners.

Particularly in southern and south-western parts of Finland there are large and thick deposits of natural post-glacial clay. Because of its origin, natural Finnish clay is very sensitive in its natural water content (50...90 %) so only the dry surface crust of the clay formations was used as a core material in the large-scale construction of earth dams that took place in Finland after the Second World War. Natural Finnish clay is so soft and sensitive that it will be liquefied by a faintest shake and thus cannot be spread and compacted into structures. But in contrast to the negative properties, it has very low permeability and good resistance to harmful chemicals. For this reason, it is potentially a good material for barriers in landfill and disposal sites.

For the control of aqueous liquids there is an international consensus that clay liner materials for landfills should have permeability equivalent to 10^{-9} m/s or less. In Finland the official requirements for a mineral barrier for ordinary waste are permeability $\leq 1 \cdot 10^{-9}$ m/s with a thickness of ≥ 1 m for ordinary waste and ≥ 5 m for hazardous waste, but Finnish clays could have permeabilities even of $8 \cdot 10^{-11}$ m/s if it was possible to compact them in the construction site.

Being a material that is widely available in the country it is less expensive than synthetic sealing products. Solve the problem of its sensitivity to handling and one would have a competitive barrier material that would more than meet the requirements of watertightness and durability. The transportation and dumping costs of the useless clay could be avoided as well.

1.2 Pilot study.

A large shopping centre in Espoo close to Helsinki was under construction in 2001. Soil in the construction site was soft clay and it was to be dumped as useless material. At the same time construction of the Kivikko landfill site in Helsinki was starting and an idea to utilize the shopping centre clay arose. If a certain amount of binding agent was mixed with the clay, perhaps its properties could be changed to be workable for compacted clay liners in the landfill site.

A study aimed at the utilization of the clay was started at the laboratory of Soil Mechanics and Foundation Engineering (SMFE) in the Helsinki University of Technology (HUT). The clay was saturated post-glacial clay with a natural water content of 50...90 % and a clay content of 40...88 %. In addition to the index properties the study included permeability tests on samples with different amounts of mixed cement. The tests were performed in a way described by Leroueil & al. (1990). Even though only a short time was available for the study, it turned out that the clay stabilized with a small amount of cement would meet the requirements of workability and permeability. After that clay stabilized with cement was selected to be the liner material.

The pilot study carried out for the project comprised only one specific type of clay and was far too modest for general conclusions. Therefore, it gave an impulse to investigate further the effect of small amounts of binders on the properties of clay particularly for protection barrier purposes, and that's why the three years project is being carried out.

The overall objective of the research project is to declare the prerequisites of Finnish clays for protection barriers as well from constructional as functional points of view. The two crucial properties for this purpose are the permeability and stiffness of clay.

Knowing that the permeability clay depends on at least the following things:

- 1) Degree of disturbance (undisturbed, remoulded, slurry).
- 2) Strain (or stress).
- 3) Types of admixtures.
- 4) Amount of admixtures.
- 5) Time and temperature. [Ravaska & al. 2003]

The aim of this Final Thesis will be the study of the permeability in stabilized with cement clay.

1.3 Mass stabilization.

Stabilization is a process to improve the strength and durability of soil. This can be reached by mixing an appropriate amount of binder agent with the soil.

Mass stabilization is a method to stabilize soft soils by adding binders in order to reduce settlements and/or to improve the stability and isolation of the land. It can be a quick and cost effective solution compared to the traditional method of pilling or mass change. Different kinds of clay, peat, sludge and soft soils can be transformed into solid layers by using this stabilization method.

This is a high environmentally compatible system as it can eliminate the need of extra aggregate and dumping waste soil off-site. We can get an entire soft stabilized stratum as a single homogeneous reinforced zone by mixing in both horizontal and vertical directions.

It is possible to combine with conventional column stabilization in hybrid solutions. These are ideal in cases where thick layers of clay are found under a stratum of peat. Column stabilization is generally the soil stabilization method of choice from a cost efficiency standpoint in locations where soft strata exceed three-five meters in thickness. Where soft soil is less than three-five meters thick, soil stabilization is the best option.

2. PROPERTIES OF NATURAL CLAY

2.1 Classification properties.

The aim of any classification system is to provide a set of common definitions which will permit useful comparisons to be made between different soils. There are a number of systems in use, but among the ones used for engineering, we will focus in the one used in Finland.

Before speaking about the classification properties, we will tell about the size of particles within a soil mass which can vary from less than 0.001 mm to 600 mm. The Finnish Standard size-ranges are given in the Table 1.

Table1. Fractional limits according the Nordic classification system. [Valtion teknillinen tutkimuskeskus, 1974]

NAME	ABBREVIATION	DEGREE	SIZE (mm)
Clay	Sa		≤0,002
Silt	Si	Fine	>0,002...0,006
		Medium	>0,006...0,02
		Coarse	>0,02...0,06
Sand	Hk	Fine	>0,06...0,2
		Medium	>0,2...0,6
		Coarse	>0,6...2
Gravel	Sr	Fine	>2...6
		Medium	>6...20
		Coarse	>20...60
Cobbles	Ki	Small	>60...200
		Big	>200...600
Boulders	Lo		>600

The term “particle” refers to an individual mineral grain within the soil mass. From the sizes given in last table it can be seen that each of the words “clay”, “silt”, etc. refers to a range of particle size. In some ways the same words are used to describe particular types of soil, since the soils are normally composed of particles

from two or more particle size-ranges. So since this moment, when we say clay we will refer to the particular type of soil which contains mainly clay sized particles.

The most important classification properties for clays are:

a) Grain Size Distribution.

Depending on the percentage of clay that the soil contains, we distinguish among:

Table 2. Terms for the designation of grain size. [Valtion teknillinen tutkimuskeskus, 1974]

NAME	% CLAY PARTICLES
Clayey silt	>10...30
Lean clay	>30...50
Fat clay	>50

b) Humus content.

Depending on the amount of organic matter that the soils have, we can identify the next sub-classification:

Table 3. Terms for the designation of organic content. [Valtion teknillinen tutkimuskeskus, 1974]

KIND IF SOIL	ORGANIC MATTER WEIGHT PERCENTAGE	NAME	SYMBOL.
Clay, Silt	≤2	Clay, Silt	Sa, Si
Clay	>2...6	Gyttja-bearing clay soils	ljSa
Silt	>2...6	Gyttja-bearing silt soils	ljSi
Clay	>6...20	Clayed Gyttja	saLj
Silt	>6...20	Silty Gyttja	siLj
Gyttja	>20	Gyttja	Lj

c) Plasticity.

Particle size analysis discloses very little about the engineering properties of very fine-grained soils. A better indication of their properties is obtained by measuring the water contents at which certain changes in the physical behaviour

can be observed. Depending upon the moisture content of a fine-grained soil, it can exist in a liquid, plastic, semi-solid or solid state, as we can see in the Figure 1.

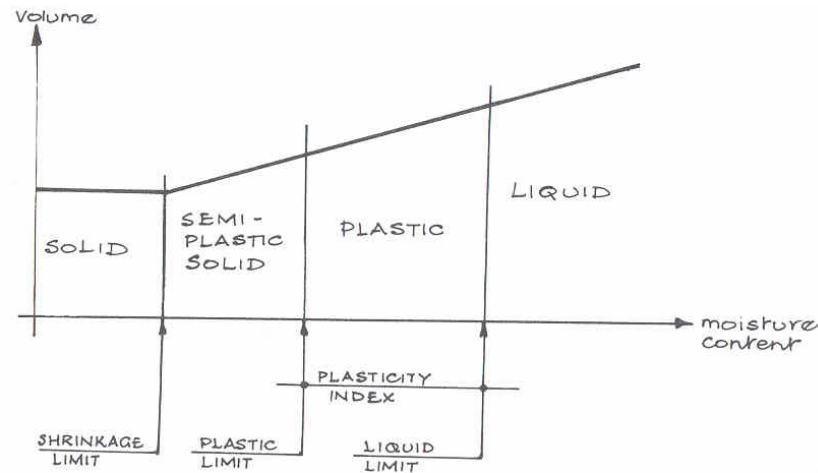


Figure 1. Relationship between soil volume and moisture content. [B. Vickers, 1987]

The *Shrinkage limit (SL)* is the water content below which no further shrinkage takes place as the soil is dried. If the water content is above SL, drying causes a loss of water without a corresponding increase in the air content of the voids, and the volume decreases. Further drying, at a water content below the SL, causes no appreciable reduction in volume, the lost water being replaced by air drawn into the voids.

The *Liquid Limit (LL)* is the minimum water content at which the soil will flow under a specified small disturbing force. The disturbing force is defined by the conditions of the test.

The *Plastic limit (PL)* is the minimum water content at which the soil can be deformed plastically, which means that the soil can be rolled into a thread 3 mm thick.

The *Plasticity index (PI)* is the range of water content over which the soil is in the plastic condition. (Equation 1)

$$PI = LL - PL \quad (1)$$

The *Liquid index (LI)* expresses the natural water content in terms of the liquid and plastic limits. (Equation 2)

$$LI = \frac{w - PL}{LL - PL} = \frac{w - PL}{PI} \quad (2)$$

where w is the natural water content. The LI varies from zero for soils at the PL to 1 for soils at the LL.

Two methods are currently employed in commercial soils-testing laboratories for the determination of the liquid limit of fine-grained soils. These are based upon using either the Casagrande apparatus or the fall-cone test. Experience has shown that the results are subject to the performance and judgment of the operator. Moreover, the Casagrande type apparatus and test method have undergone many small but significant variations since it was first proposed by Casagrande in 1932. These variations give rise to differences in the values of the liquid limit determined from the test. The fall-cone method is the preferred method of determining the liquid limit of a soil, and the one we are using in the laboratory. [CEN ISO/TS 17892-12:2004 (E)]



Figure 2. Fall-cone test.

Essentially the test lies in measuring the moisture of the sample when the cone penetrates the following depths. Normally in Finland we use the 100 gm and 30 degrees cone for natural clay, 60 gm and 60 degrees cone for remoulded clay, and 10 gm and 60 degrees cone for remoulded clay as well, but it is not always like

this, it depends on each particular case, although the cone will have to penetrate 10 mm in all of them to get the Liquid Limit.

Table 4. Cone penetration requirements. [CEN ISO/TS 17892-12:2004 (E)]

	80 g/30°	60 g/60°
initial penetration	about 15 mm	about 7 mm
penetration range	15 to 25 mm	7 to 15 mm
maximum difference between two successive tests	0,5 mm	0,4 mm
w_L determined from penetration of:	20 mm	10 mm

The best way to get this value would be testing at least three samples with different water content and checking the depth at which the cone sets. Drawing these results we could link them by a straight line, so we could know the water content corresponding to the 10 mm depth (60 gm/ 60° cone).

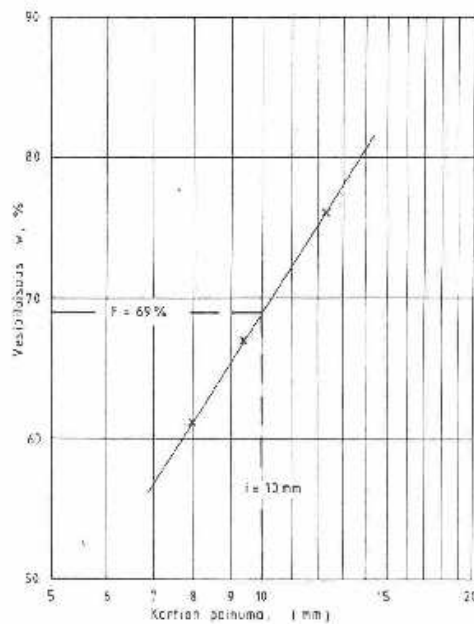


Figure 3. Determination of cone liquid limit. [GLO-85]

But this method is very hard to use, so finally we proceed by using tables as for instance the Table 5.

Table 5. Setting the Liquid Limit. [GLO-85]

Cone liquid limit Undrained shear strength Undisturbed sample Disturbed sample Factor α Coarse silt, fine sand Silt Clay Mud

Hienosäilykku		Teikkauslujuus		Värsälujuus (σ)			
$F = a \cdot w(\%)$		$\sigma_k = \frac{9,80555 \cdot R_2}{35 + 1,064 \cdot R_2} \text{ (kN/m}^2\text{)}$		Karkea siltti, hieno hiekkasiltti $F < 30$	Keskisiltti, hienosiltti $F=30-38$	Silt $F=38-40$	Liiju $F > 40$
Lujuus-luku R	60 g/60 ^o karkon painuma l [mm]	Häiritön näytteen leikkauslujuus, σ_k [kN/m ²]	Häirityn näytteen leikkauslujuus, σ_{kr} [kN/m ²]				
13,8	8,5	2,28	2,45	1,03	1,04	1,07	1,12
13,5	8	2,23	2,45	1,03	1,04	1,06	1,11
13,2	7	2,18	2,35	1,02	1,03	1,06	1,10
12,8	8	2,14	2,26	1,02	1,03	1,05	1,09
12,5	8	2,10	2,26	1,02	1,03	1,05	1,09
12,2	9,5	2,06	2,16	1,02	1,03	1,04	1,08
11,9	1	2,02	2,16	1,02	1,02	1,04	1,07
11,7	2	1,98	2,06	1,01	1,02	1,03	1,06
11,5	3	1,94	2,06	1,01	1,02	1,03	1,05
11,2	4	1,90	1,96	1,01	1,02	1,02	1,04
11,0	9,5	1,86	1,96	1,01	1,01	1,02	1,04
10,8	5	1,82	1,86	1,01	1,01	1,02	1,03
10,6	7	1,78	1,86	1,01	1,01	1,01	1,02
10,4	8	1,75	1,86	1,00	1,01	1,01	1,02
10,2	9	1,71	1,77	1,00	1,00	1,00	1,01
10,0	10,0	1,67	1,77	1,00	1,00	1,00	1,00
9,8	1	1,64	1,77	1,00	1,00	1,00	0,99
9,6	2	1,61	1,67	1,00	1,00	0,99	0,98
9,4	3	1,58	1,67	1,00	0,99	0,99	0,97
9,25	4	1,55	1,67	1,00	0,99	0,98	0,96
9,05	10,5	1,52	1,57	0,99	0,99	0,98	0,96
8,90	5	1,49	1,57	0,99	0,99	0,97	0,95
8,75	7	1,46	1,57	0,99	0,98	0,97	0,94
8,60	8	1,43	1,47	0,99	0,98	0,97	0,93
8,40	9	1,40	1,47	0,99	0,98	0,96	0,93
8,25	11,0	1,37	1,47	0,99	0,98	0,96	0,92
8,10	1	1,34	1,47	0,98	0,97	0,95	0,92
7,95	2	1,32	1,37	0,98	0,97	0,95	0,91
7,80	3	1,30	1,37	0,98	0,97	0,95	0,91
7,65	4	1,28	1,37	0,98	0,97	0,94	0,90
7,55	11,5	1,27	1,37	0,98	0,96	0,94	0,90
7,40	6	1,25	1,27	0,98	0,96	0,93	0,89
7,30	7	1,23	1,27	0,98	0,96	0,93	0,89
7,15	8	1,21	1,27	0,98	0,96	0,93	0,89
7,05	9	1,19	1,27	0,98	0,95	0,93	0,88
6,95	12,0	1,17	1,18	0,97	0,95	0,92	0,88
6,80	1	1,15	1,18	0,97	0,95	0,92	0,87
6,70	2	1,13	1,18	0,97	0,95	0,92	0,87
6,60	3	1,11	1,18	0,97	0,94	0,91	0,85
6,45	4	1,09	1,18	0,97	0,94	0,91	0,86
6,35	12,5	1,07	1,08	0,97	0,94	0,91	0,85
6,25	6	1,05	1,08	0,97	0,94	0,90	0,85
6,15	7	1,03	1,08	0,96	0,93	0,90	0,84
6,10	8	1,01	1,08	0,96	0,93	0,90	0,84
6,00	9	0,99	1,08	0,96	0,93	0,90	0,84
5,90	13,0	0,98	1,08	0,96	0,93	0,89	0,83

We go into the table with the depth of the cone penetration and with the type of soil, so we will know the a value. After that we have to get the water content putting the sample into the oven (105° C), and finally, $F(\text{Liquid Limit}) = a \cdot w(\%)$.

2.2 Shear strength.

2.2.1 Effective shear strength parameters.

The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles or in masse upon the action of a tangential stress. It is the stress that has to affect a soil in order to cause its failure, an hydrostatic state would compress it but it would never bring the soil to the failure.

The shear strength in clays depends a lot on the drainage conditions, so the shear process of cohesive soils is more complicated than with sands because of the usual pore water present.

Most practical shear-strength theory of soil is based upon the Mohr-Coulomb failure criterion. Coulomb showed that the shear strength τ of a soil can be expressed by

$$\tau = c + \sigma_n * \tan \varphi \quad (3)$$

where c and φ are shear-strength parameters and σ_n is the normal stress acting on the failure surface.

It is important to realise that c and φ are not merely parameters defining the equation. c and φ are physical properties of the material. c , the cohesion is constant and φ , the friction angle can vary depending on the situation.

Using Terzaghi's concept of effective stress, $\sigma_n' = \sigma_n - u$, where σ_n = total normal stress, u = pore-water pressure and σ_n' = effective normal stress, Coulomb expression can be written as

$$\begin{aligned} \tau &= c' + (\sigma_n - u) * \tan \varphi' \\ \tau &= c' + \sigma_n' * \tan \varphi' \end{aligned} \quad (4)$$

where c' and φ' are parameters related to effective stress.

2.2.2 Undrained Shear Strength.

Undrained shear occurs in practical problems whenever external loads change at a rate much faster than the rate at which the induced pore pressures can dissipate. That excess pore pressures dissipate relatively slowly from a clay and relatively rapidly from a sand. The condition of undrained shear is thus of great practical importance in the case of clays.

Undrained conditions imply that the soil is sheared at a constant moisture content and, if the soil is fully saturated then, there is therefore no volume change during the process.

When clay is subjected to shear, and the drainage of the water from the voids of the clay soil does not take place effectively, the shear is independent of the normal stress because of the pore water. If we apply the clay by a normal stress σ_n , the induced pore water pressure u will be equal in magnitude to the normal stress and so $\sigma' = \sigma_n - u = 0$. In this case the shear strength of the clay will be $\tau = c = c_u$ (*Undrained shear strength*) which analytically represents a straight line parallel to the σ_n -axis and at a distance c from the latter.

2.2.3 Sensitivity.

Consider a small element of soil, subjected to a uniform normal stress and a variable shear stress, Figure 4 shows a typical relationship between shear stress and shear strain for such an element of real soil.

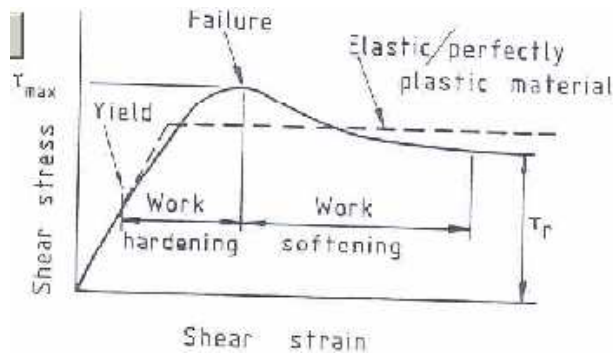


Figure 4. Stress/strain relationships: typical real soil (solid line), and elastic/perfectly plastic model soil (broken line). [C.R. Scott, 1980]

For very small values of the shear stress, the corresponding strains may be nearly linear and elastic. As the shear stress increases, a point is reached where significant plastic shear strains starts to develop. At this point the material is said to *yield*. The resistance of the soil to increasing plastic shear strains is called the *shear strength*. At first, the plastic strains are strictly limited, because they result in an increased resistance to further deformation. The material is said to work harden. The yield is said to be stable, and unlimited volumetric strain cannot occur. However, strain hardening can only increase the resistance to shear stress to a strictly limited extend. Once the applied stress exceeds the limiting value τ_{\max} , shear strains increase continuously for as long as the shear stress is maintained. The yield is to said unstable and the soil is said to fail. In some soils, and under some loading conditions, the shearing resistance decreases after failure. This decrease is described as strain softening. [C.R. Scott 1980]

For many soils there is a great difference between the peak shear strength of the soil after it has been remoulded without change of the water content. The ratio of the peak shear strength of the clay strength τ_u to the maximum value of the remoulded shear strength of the same clay τ_r is called the coefficient of sensitivity, s , of a clay,

$$s = \frac{\tau_u}{\tau_r} \quad (5)$$

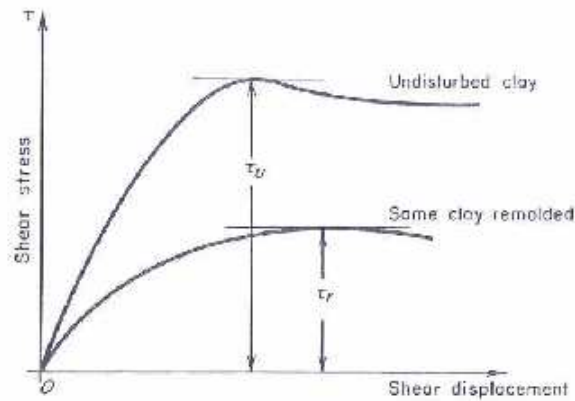


Figure 5. Sensitivity of clay. [C.R. Scott, 1980]

So, the shear strength of the soil depends very much upon whether its natural structure is disturbed or not, and it happens because during the remoulding, most of the effective stress which had been carried by the mineral skeleton is transferred to the pore water and also because the bonds between grains are broken.

2.3 Settlement Properties.

Settlement is caused by application of increased stress on the soil. Before imposing a load on the soil, the soil has come to some sort of equilibrium, in which the soil particles can bear the weight of the overlying soil.

On applying an increased load to the soil through a footing, the soil is subjected to increased stress levels, which cause the particles to rearrange in a denser structure that can accommodate these higher stresses. The volume of voids between the soil particles decreases, and this decrease is reflected in settlement at the soil surface.

Of course, this rearrangement is only caused by changes in the stresses that the soil skeleton feels - the effective stresses, rather than the change in total stress, which includes pore water pressure. So, to compute the settlement of a soil, the effective stresses in the soil before and after application of the new load must be determined.

For sands, the rearrangement of soil particles is virtually instantaneous, whether the sand is above or below the water table. If it is below, the sand structure is so open that water is not impeded from being squeezed out as the sand grains move to their denser structure. Sands also respond elastically to applied stress (as long as it is not excessive). If load is taken off, the sand structure will simply revert to its previous looser state.

For clays, the settlement is not instantaneous, because water travels slowly through a clay structure - voids are not continuous, and there are electrochemical attractions etc. There is some (small) component of elastic rearrangement; however, the bulk of the volume change occurs over weeks, months or years as the water is displaced from the more highly stressed areas. This rearrangement of particles is commonly called consolidation.

Incidentally, with both sands and clays, this rearrangement of particles to a denser structure is accompanied by an increase of soil strength.

2.4 Permeability.

Permeability is a measure of the rate at which fluid passes through a porous medium. In the case of water passing through soil, Darcy defined a coefficient of permeability k as in the Equation 6.

$$v = -k \cdot \frac{dh}{dl} = k \cdot i \quad (6)$$

where v is the apparent velocity of flow, and is equal to the average rate of flow of water across unit area in the soil. The i value is the hydraulic gradient and it is equal to the average rate of flow of water across unit area in the soil. The permeability is almost independent of this.

The permeability has a dimension of velocity.

Darcy's law states that the coefficient of permeability is independent of the hydraulic gradient which is right mainly in sandy soils. However, within clayey soils

Darcy's law is not always valid because the permeability depends on the hydraulic pressure. Under a limit value of the hydraulic gradient, the permeability is fairly close to zero in the area called pre-linear Darcy's law.

So the permeability depends on the soil type and the conditions in the soil will be different if the soil is of coarse or fine grained.

According with C.R. Scott and B. Vickers, the coefficient of permeability varies with:

- The density and viscosity of the soil water. The viscosity of the water passing through the soil is largely affected by the temperature. Under normal conditions there is little temperature variation of the soil water on site below about 2 m below ground level.
- The shape and arrangement of the soil particles. The particles within a soil mass can be arranged in a variety of ways. Figure 6 shows how tortuosity of the path of a particle of water can be varied according to the shape and the arrangement of the particles.
- The porosity of the soil. The arrangement of the particles this affects the void ratio e of the soil and since the porosity n of the soil and the unit weight γ are dependent upon the void ratio, it is clear that alteration of either the void ratio or the unit weight increases or decreases the permeability according to the change in arrangement of particles.
- The turbulence of flow.
- The mineral characteristics of Fine-grained soils. Different types of particles attract and hold different thicknesses of absorbed water around each particle, and consequently reduce the effective pore size.
- The degree of saturation. If the soil is about 85% saturated, air occurs as bubbles within the pore space of the soil and these can block the seepage channels and thus reduce the permeability of the soil. Under these conditions

Darcy's law is only approximately valid. It is important to note that a change of 1% in the degree of saturation above 85% level can change the permeability by about 3%. Below a degree of saturation of 85% Darcy's law is invalid because continuous air (rather than bubbles).

- The thickness of the adsorbed layers, in the case of fine-grained soils.
- The stratification or layering. Due to the manner in which soils are deposited, they are usually stratified or layered. Consequently a soil can have markedly different permeability values in horizontal and vertical directions. This effect is known as anisotropy. Generally the permeability value in a horizontal direction (along layers) is much greater than the permeability value in a vertical direction (across the layers).
- Others like the presence of organic matter, sand lenses, silt intrusions, fissures, etc.

Depending on the kind of soil; the permeability of natural soil has typical values [C.R. Scott, 1980]. These values are:

	m/s
Gravel	$k > 10^{-2}$
Clean sand	$10^{-2} > k > 10^{-5}$
Silt	$10^{-5} > k > 10^{-8}$
Fissured clay	$10^{-4} > k > 10^{-8}$
Intact clay	$k < 10^{-8}$

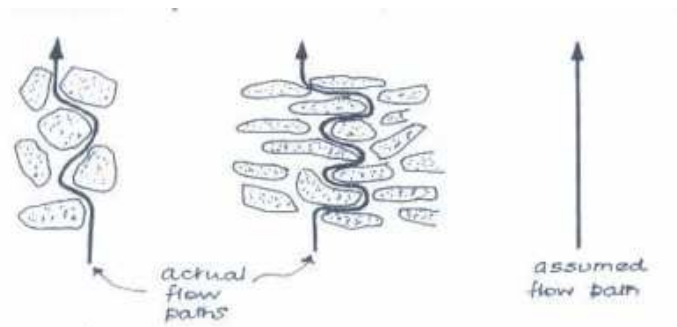


Figure 6. Different flow paths. [B. Vickers. Laboratory work in civil engineering soil mechanics, 1978]

3. PREPARATION AND PRESERVATION OF STABILIZED SAMPLES

3.1 Introduction.

What we are going to expound in this section of the report is the process that we have been carrying out in order to prepare the stabilized samples to which we have subjected our tests.

3.2 Disturbing and stabilizing the clay.

First of all we have to take the core samples from the cold room. They have been there at a temperature between 5 and 7°C and 100% of humidity since they were got in the field weeks or months ago. They can be there and keep their own properties because the cold room simulates the conditions at which the soil is in the field.

The samples are in a cylindrical shape into a cylindrical and metallic tube. We extract them by being helped by a machine like the one showed in the Figure 6.a, so after that we have a cylindrical clay sample as we can see in Figure 6.c that keeps its shape because we are working with clay.



Figure 6.a. Putting the sample from the cold room into the machine that helps us to extract the clay from the metallic tube.

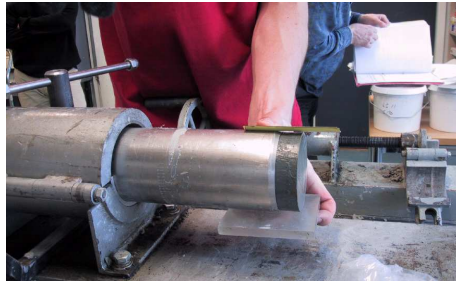


Figure 6.b. Extracting the clay.



Figure 6.c. The clay sample after taking it out from the metallic ring.

Once we have the clay sample, we put it in a bucket and we remould it with a mixer like one in the Figure 7.b. We can see the way of doing it in the Figure 7.a.



Figure 7.a. Remoulding the sample in the bucket.



Figure 7.b. Mixer.

As we can see, the clays are very sensitive. After remoulding they become very soft, much more than they were in the field, so we made the fall-cone test in

order to know the undrained shear strength of the remoulded clay and compare the value with the one of the undisturbed sample that we got before. We can see the comparison in the Figure 8.a. and Figure 8.b.



Figure 8.a. Getting the undrained shear strength of the undisturbed sample with the fall cone test.



Figure 8.b. Getting the unconfined shear strength of the remoulded sample with the fall-cone test by using the 10gm/60° cone because of the softness of the soil.

After that we take a quantity of clay from the bucket and we put it in a plastic bag like in the Figure 9.



Figure 9. Putting the remoulded clay in a plastic bag.

And then we put a quantity of cement. This quantity depends on the amount of clay in order to get an exact percentage of cement respect the dry weight of the soil.



Figure 10. Putting the cement into the plastic bag.



Figure 11. Clay and cement into the plastic bag.

Once both materials are into the plastic bag we close it and we mix the content with our own hands during five minutes. We can notice how fast it becomes stronger while we are mixing them.



Figure 12. Mixing the materials.

The result is a mixture like the one we can see in the Figure 13.



Figure 13. Materials into the plastic bag after five minutes mixing them.

Sometimes we want to know the fall-cone undrained shear strength of the mixture just when it has been prepared so we take a cup and fill it with the material. After that we test it in the fall-cone test.



Figure 14.a. View 1 making fall-cone test to the stabilized clay in order to get the undrained shear strength.



Figure 14.b. View 2 making fall-cone test to the stabilized clay in order to get the undrained shear strength.

3.3 Unconfined compression test.

The steps we have already done are common to all the tests when we use stabilized samples, but from this point the way of proceeding depends on the

experiment we will make, so we will start defining the way to prepare the unconfined compression test samples.

In this case we will have to prepare at least five samples because as we said we have to test them at the first, seventh, fourteenth, twenty first and twenty eighth day after stabilizing the clay. In some cases we have been preparing more than five samples in order to get different results each day and be able to compare them.

For the undrained shear strength test we need cylindrical samples 20 mm diameter and 40 mm high, so we fill at least five syringes with these dimensions with the stabilized clay. Normally we fill them with a spatula, but sometimes the clay has become so consistent that we have to proceed in the following way.

We take the stabilized clay from the plastic bag and we put it in a container like the one in the Figure 15. After that we press it against a metallic plate and introduce the syringe into the hole that has been created, and then we have already the sample, as we can see in the Figure 19.



Figure 15. Container that helps us to fill the syringes with the stabilized clay.



Figure 16. Filling the container with stabilized clay to make the samples.



Figure 17. Pressing the container and the metallic plate.



Figure 18. Putting the syringe into the hole.



Figure 19. We have the sample.

The following step is to put the samples in packs of three samples in a plastic bag and put it in the camera where they will be at a temperature between 23.3 and 24.2°C until they have to be tested.



Figure 20. Plastic bag with three samples into the cupboard at a temperature between 23.3 and 24.2°C.

In this moment the samples are ready and we will take them when we have to test them.

3.4 Oedometer test.

If we are testing the stabilized samples with the oedometer apparatus instead of the trail one the samples are not the same, so the way of preparing them is different.

In this case we take the stabilized clay from the plastic bag and we put it into an oedometric ring like the one we can see in the Figure 21.

There is a friction between the soil and the ring, what alters the natural settlement of the soil. In order to make this friction as small as possible we spread silicone in the inner surface of the ring. By doing this the permeability of the porous stones decrease as the test proceeds because their voids become blocked. This problem is solved by boiling the porous stones before testing. Once we have spread the silicone we put the soil inside the ring by being helped by a spoon, the clay has to be very well arranged into the specimen.



Figure 21. Oedometric ring.



Figure 22.a. Putting the clay into the ring, 1.



Figure 22.b. Putting the clay into the ring, 2.



Figure 22.c. Putting the clay into the ring, 3.



Figure 22.d. Putting the clay into the ring, 4.

The result is a specimen like the one in the Figure 23.



Figure 23. Oedometer ring sample.

The purpose we are using the rings for is to test clay permeability, thus the dimensions of the must agree with this purpose. They are the following:

Height of the ring	h	15 mm to 80 mm
Diameter of the ring	d	45 mm to 250 mm
Relationship	d/h	3 to 5

We have been stabilizing the clay and putting it directly into the ring, but we have observed that proceeding in this way the samples are not saturated when we make the oedometer and permeability test, so what we are starting to do for next tests is to keep the sample into the water for seven days and see if we can get the sample saturated. We don't have to forget that one of the Terzaghi's conditions to use his unidimensional consolidation theory is that the soil must be saturated.

4. UNDRAINED SHEAR STRENGTH

4.1 Field vane test.

The vane test gives a rapid estimation of the undrained shear strength of fine-grained soils and can be of use in certain field situations. The vane is available in two forms, one appropriate for laboratory testing and the other for field or *in situ* testing. The laboratory vane, which is little used in practice, is essentially a small-scale version of the field vane.

Sampling is not involved in *in-situ* testing that means that the problems and doubts associated with possible disturbance are obviated. Furthermore, the limitations imposed on the accuracy of the laboratory modelling by the relatively small samples can be reduced or eliminated. The field vane test is particularly useful in soft, sensitive soils where there may be excessive disturbance on sampling or, indeed, where sampling cannot conveniently be undertaken. It can be performed in virgin ground or in the base of a trial pit or borehole.

The vane consists of four or six blades set at right angles, attached to a central circular rod as we can see in the Figure 24. The vane blades are usually manufactured from high-quality stainless steel and usually have a height-to-diameter ratio of 2:1. Common sizes are 150 mm long x 75 mm diameter and 100 mm long x 50 mm diameter. The blades should be thin and should have a cutting edge on their lower edge.

The area ratio is usually less than 12% and is given by the expression

$$Area_ratio(\%) = \frac{8t(D-d) + \pi d^2}{\pi D^2} * 100 \quad (6)$$

t = thickness of blade (mm)

d = diameter of vane rod (mm)

D = overall diameter of blades when rotated (mm)

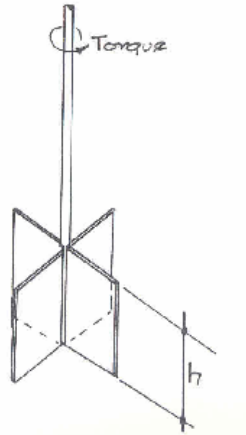


Figure 24. Vane test. [B. Vickers, 1978]

The purpose of these conditions is that both remoulding and disturbance effects are kept sensibly minimal.

During the test the vane is pushed into the soil until it is totally embedded and a torque is applied to the circular rod and is steadily increased. Failure of the soil is noted by a reduction in the torque required; it is assumed that the soil fails on a cylindrical surface area enclosing the extremities of the four blades as we can appreciate in the Figure 25.

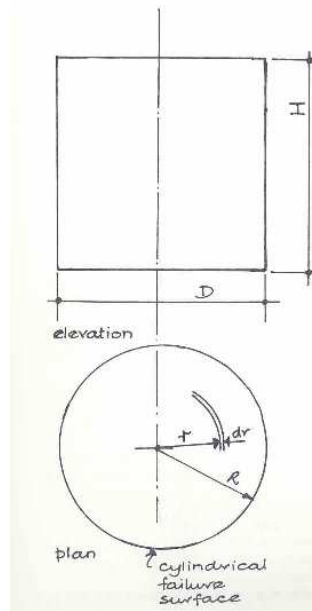


Figure 25. Cylindrical failure surface. [B. Vickers, 1978]

The test should be performed with the top of the vane blades at least 0,5 m below the base of the borehole or trial pit so as to ensure that the soil being tested has not been disturbed significantly by boring or excavation operations.

To maintain vertically down the borehole, steady bearings may need to be fitted to the vane rods. Some vanes are also fitted with a protective shoe from which the vane is pushed when it has penetrated to the required depth. This is particularly useful when the vane is introduced directly into the soil from ground level.

The angular rate of rotation is usually of the order of 6° per minute. [B. Vickers, 1978]

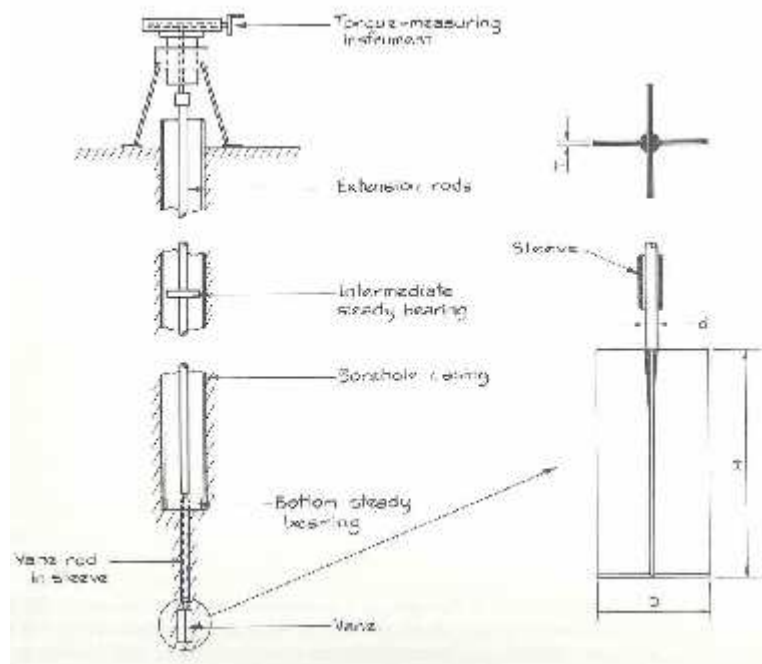


Figure 26. General arrangements of the field vane test. [B. Vickers, 1978]

The physical dimensions of the vane are recorded, together with a maximum torque achieved during shear. In the laboratory, maximum torque is usually calculated from a record of the maximum horizontal deflection angle of a spring attached to the circular rod; in *in-situ* testing a torque wrench or similar device records the failure value. Assuming that the distribution of shear strength around the cylindrical surface area and around the circular ends is uniform, the torque T at failure is given by the

following expression, where C_u is the undrained shear strength, H is the height of vane and D is diameter of vane.

$$\begin{aligned}
 T &= C_u * \text{CylindricalSurfaceArea} * \text{LeverArm} = \\
 &= C_u * \int_0^R 2\pi r * dr * r * (2\text{No.ends}) = \\
 &= C_u \pi D H * \frac{D}{2} + 4\pi C_u \left(\frac{r^3}{3} \right)_0^R = C_u \left[\frac{\pi D^2 H}{2} + \frac{4\pi}{3} (R^3) \right] = \\
 &= C_u \left(\frac{\pi D^2 H}{2} + \frac{\pi D^3}{6} \right) = \pi D^2 C_u \left(\frac{H}{2} + \frac{D}{6} \right) \quad (7)
 \end{aligned}$$

and

$$C_u = \frac{T}{\pi D^2 \left(\frac{H}{2} + \frac{D}{6} \right)}$$

[B. Vickers, 1978]

4.2 Unconfined compression test.

4.2.1 Objective.

Because it was planned in the three years project, our objective in this part of the global project is to know the minimum amount of cement the clays would have to be stabilized with in order to use them as barriers and other uses in civil engineering. That is the minimum amount of cement in order to achieve a minimum c_u value. Thus, we tested different samples with different amount of cement, 2%, 4%, 5% in our laboratory.

4.2.2. Unconfined compression test theory.

As it is known, the failure of a saturated soil is represented by a horizontal line in the Mohr-Coulomb's theory graph if we work with total stresses. So, if the soil can be assumed to be fully saturated, and this is commonly the case with clay soils, except

in arid climate, only one parameter (C_u) remains to be determined to have the shear strength information about the soil. Only one test is therefore required, and this may be, most easily carried out in unconfined compression (that is, with $\sigma_3 = 0$), because if $\sigma_3 = 0$ then the Mohr's circle always goes through the origin when total stresses are plotted, and σ_1 is known with the test, so we can obtain C_u .

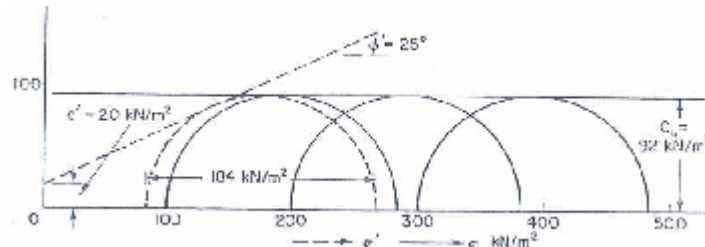
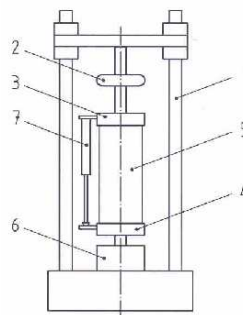


Figure 27. Shear stress/Normal stress in a fully saturated soil with no drainage. [B. Vickers, 1978]

A loading machine for performance of unconfined compression test normally consist of the following main parts:

- Top and bottom platen between which the soil specimen is placed,
- load frame with a drive unit to compress the soil specimen (loading press),
- load measuring device to measure the force applied to the soil specimen,
- compression measuring device to measure the axial compression of the specimen.

All of them are represented in the Figure 28.



Key

1 Load frame	5 Soil specimen
2 Load measuring device	6 Drive unit
3 Top platen	7 Axial compression measuring device
4 Bottom platen	

Figure 28. Schematic drawing of a loading machine for performing unconfined compression tests. [B. Vickers, 1978]

This test may be made either in the triaxial apparatus with zero cell pressure, or in a specific apparatus made for the purpose. The first option is the one we have been using in our laboratory. The apparatus is shown in the Figure 29.

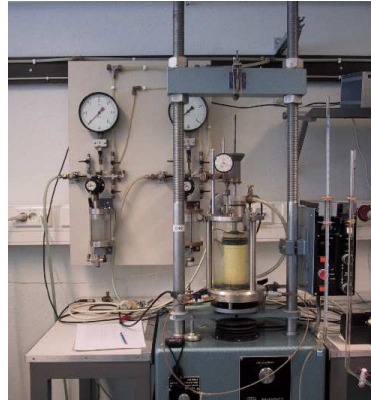


Figure 29. Apparatus used in our laboratory.

The unconfined compression test is only applicable to materials which will stand unsupported and have fairly low permeability so that undrained conditions effectively exist over the duration of the test. It is specially a good test for stabilized soil.

4.2.2 Test procedure.

4.2.2.1 General.

The cross-sectional area of the specimen may either be circular or square, and shall be at least 1000mm^2 . In our case, we use circular cross-sectional area.

For cylindrical specimens the ratio between height and diameter shall be between 1, 8 and 2, 5. We use 20 mm diameter and 40 mm diameter approximately, it allows us the possibility to make tens or hundreds of samples in one day with small amounts of soil.

The largest particle in the specimen should not exceed $1/6$ of the specimen diameter for cylindrical specimens and not exceed $1/6$ of the specimen diameter.



Figure 30. Unconfined compression test samples still inside the syringe.

4.2.2.2 Preparation of remoulded specimens.

Unless otherwise specified, remoulded specimens shall be prepared by remoulding undisturbed material at its natural water content without significantly changing the water content.

After remoulding the soil shall be kneaded, still without changing the water content, into the mould for making remoulded specimens. Care shall be taken to avoid entrapped air in the material during the kneading process.

Loss of the water during the remoulding process and evaporation during the succeeding remoulding process may be minimized by wrapping the material in a thin rubber membrane or in a plastic bag and kneading thoroughly with the fingers to assure complete remoulding.

4.2.2.3 Compression.

The specimen shall be placed in the loading machine so that it is centred with respect to the bottom platen. If the top platen can tilt, the specimen shall be centred with respect to the top platen.

The load measuring device shall be zeroed when there is no contact between the top of the specimen and the top platen. The loading press shall be adjusted so that the top platen just makes contact with the specimen. The displacement measuring device shall then be zeroed and the corresponding reading on the load measuring device recorded.



Figure 31. The displacement measuring device (left) is zeroed at the beginning of the test and the corresponding data on the load measuring device (right) is recorded.

A platen rate shall be selected. The compression of the specimen at the selected rate shall be started. We record the values of load and displacement.

In this test a load applies the sample in a magnitude that produces a constant deformation of the sample. We report the data when the vertical displacement of the sample changes in 0.05 mm, as we can see in the Table 6, and we will stop when we notice the failure of the sample.

Table 6. Table to fill while doing the unconfined compression test.

Test (Start) date:	07.06.2006/mh		korkeuden	eps1	korjattu	voima	voima
Compression date:	14.06.2006/mh		muutos:	%	ala	lukema	
Site:	HUT (Ossinlampi)		[mm]		m2		kPa
Piston:	2006 Norj P3						
Depth (m):	1,4-1,92m		0	0,00	0,000341	0	0
Before test			0,05	0,12	0,000341	0,77	22,13136
Admixture:	Cement		0,1	0,25	0,000342	1,69	48,51416
		kg/m					
sideainemäärä:	44,8	3	0,15	0,37	0,000342	2,16	61,92974
Sample temp:	22	C*	0,2	0,49	0,000342	2,82	80,75283
Sample age (day):	7	d	0,25	0,62	0,000343	3,47	99,24315
Sample height (mm):	40,62	mm	0,3	0,74	0,000343	4,02	114,8309
Sample							
diameter(mm):	20,83	mm	0,35	0,86	0,000344	4,59	130,9503
Sample area(m2):	0,0003408	m2	0,4	0,98	0,000344	4,98	141,9004
Sample weight (gm):	21,74	g	0,45	1,11	0,000345	5,47	155,6688
Sample no:	CB-III (1)		0,5	1,23	0,000345	5,7	162,0123
Load ring	1		0,55	1,35	0,000345	5,88	166,9202
		mm/					
Strain rate:	0,4	min	0,6	1,48	0,000346	6,09	172,666
Piston weight:	0	kg	0,65	1,60	0,000346	6,27	177,5473
		g/cm					
Bulk density	1,57	3	0,7	1,72	0,000347	6,44	182,133
water content :	67,96	%	0,75	1,85	0,000347	6,54	184,7295
			0,8	1,97	0,000348	6,55	184,78
After test			0,85	2,09	0,000348	6,5	183,1392
Cup number	185		0,9	2,22	0,000348	6,42	180,6578
Cup wt. (gm)	45,55		0,95	2,34	0,000349	5,95	167,2213
Cup+Wet sample							
(gm)	67,2		1	2,46	0,000349	5,55	155,7829
Cup+Dry sample (gm)	58,44		1,2	2,95	0,000351	5,11	142,7085
Wet sample (gm)	21,65		1,3	3,20	0,000352	4,57	127,304
Dry sample (gm)	12,89						
Water content%	67,96						
Sample vol (cm3)	13,84						
Specific gravity	2,75						
Void ratio	1,95						
Dry density (gm/cm3)	0,93						
Degree of saturation							
Sr %	95,7						
Cement ratio Aw(%)	5						
initial W/C ratio	15,04						

Figure 32. Carrying out the unconfined compression test.

The compression may be stopped when the vertical strain reaches 15%, or starts to decrease, whichever is earlier. It can be also stopped because some fissures appear on the sample what will be usual when we are testing stiff samples like the ones stabilized.



Figure 33. Failure of a sample in the unconfined compression test.

4.2.2.4 Dismounting.

The axial load shall be removed.

A rough sketch shall be made or photograph shall be taken of the specimen indicating the failure planes.

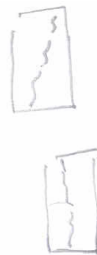


Figure 34. Drawing the specimen after the failure.

The water content, if required, shall be determined as quickly as possible after the compression test, from a representative part of the soil specimen.

The specimen shall be broken into pieces and the soil shall be described. It shall be noted, if there are particles greater than permitted.

4.2.2.5 Testing times.

The samples tested for this report were cement stabilized samples, and they were tested the first, the seventh, fourteenth, twenty first and twenty eighth day after making the mixture.

4.2.3 Test results.

With the data we get from the unconfined compression tests, we are able have the following results.

1) q - ε_1 graphic.

We define $q = \sigma_1 - \sigma_3$, where σ_1 is the normal tension in the direction in which we apply the load, and σ_3 would be the normal tension in the radial direction of the sample. As we explained before σ_3 is zero testing the unconfined compression test, so in this case $q = \sigma_1$.

$$\varepsilon_1 \text{ is defined as } \varepsilon_1 (\%) = \frac{\Delta h}{h_0} * 100.$$

The values we directly report from the unconfined compression tests are the height of the sample and σ_1 , so in this case we can use as an example the test carried out for the sample 4670kss(c).

The test has the following results:

Height (mm)	σ_1 (kPa)
0	1,436113
0.1	16,33007
0.2	22,00371
0.3	24,51278
0.4	26,15581
0.5	26,93935
0.6	28,00136

0.7	28,4933
0.8	29,26371
0.9	29,74908
1	30,23153
1.1	30,43187
1.2	30,90922
1.3	31,10591
1.4	30,74713
1.5	30,66609
1.6	30,86059
1.7	30,50401
1.8	30,14889
1.9	29,52187
2	28,89778
2.1	28,82039
2.2	27,92952
2.3	27,04303

So we can draw the next the Figure35.a.

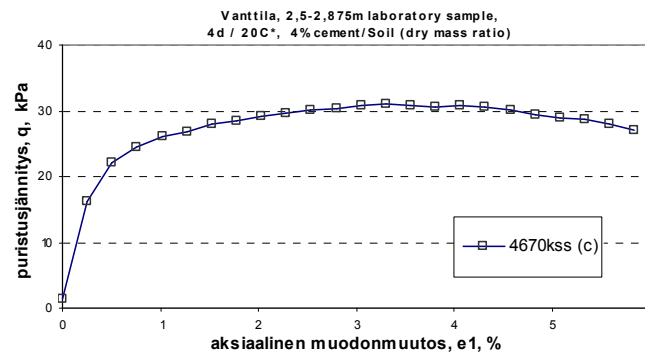
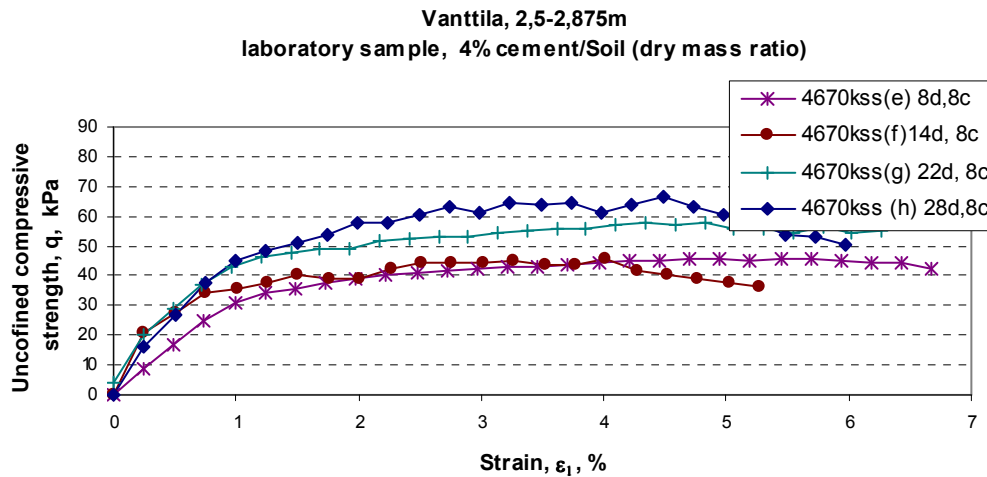


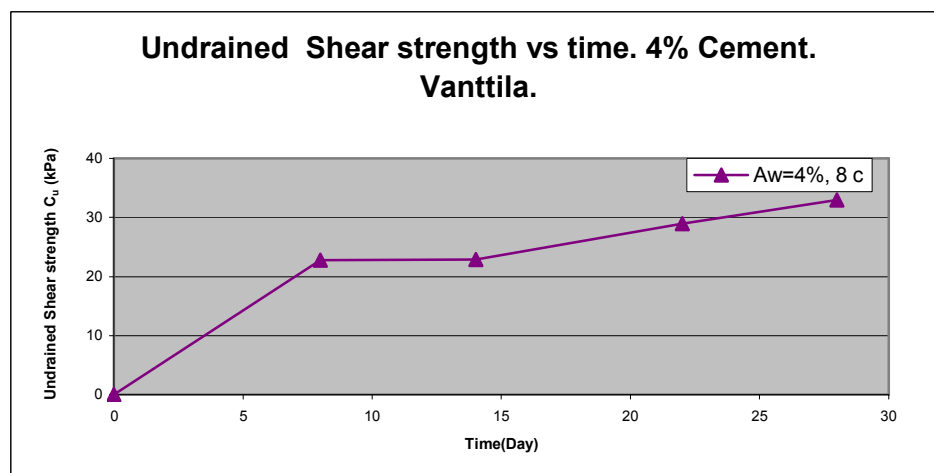
Figure 35.a. Unconfined Compression Test results for 4670kss(c) sample.

By changing the variables of the test with the same sample, for example testing in different days, we can observe the consequences of different test variables, in this case the time of testing. In the Figure 35.b we can appreciate this. We can see how the testing time influences the results. This Figure has been obtained with the sample 4670kss.

Figure 35.b. q - ϵ_1 graph.2) C_u - t (time) graphic.

By using the same results as in the figures before, we can observe how the C_u value depends on the time. We could already guess it by looking at the Figure 35.b.

It is known that $\tau = q/2 = \sigma_1/2$, and the maximum value $\tau_{\max} = C_u$. So if we record the maximum value of τ of several unconfined compression tests carried out different days we can achieve graphs like for instance the one in the Figure 36.

Figure 36. C_u - t (time) graphic.

4.3 Fall-cone test.

4.3.1 Introduction.

In the Scandinavian countries the undrained shear strength of clay is usually investigated by means of the fall-cone test, often in combination with the unconfined compression test. The fall-cone test was developed by the Geotechnical Commission of the Swedish State Railways between 1914 and 1922. Compared with other methods of investigation the fall-cone test is extremely simple and that is why it has therefore gained a wide use in Scandinavia.

The used apparatus is shown in the Figure 37.

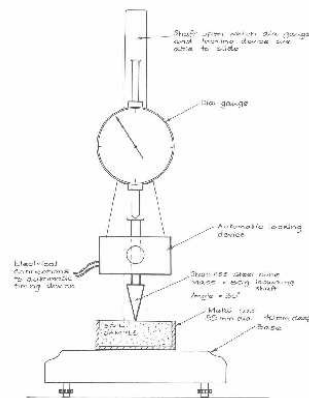


Figure 37. Cone penetrometer. [S. Järnvägar, 1922]

The test is carried out as follows. A metal cone is placed vertically with its apex just in contact with the smooth and top surface of the clay sample, as we can see in the Figure 38.

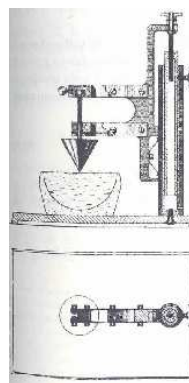


Figure 38. Fall-cone apparatus, correct position of the cone. [S. Järnvägar, 1922]

The cone is dropped freely into the clay and the depth of penetration measured. A lighter or blunter cone shall be selected if the cone penetration is more than 13 mm, and a heavier or sharper cone shall be selected if the penetration is less than 4 mm. For undisturbed samples at least three tests shall be performed. If any value deviates more than 10% from the average, an additional test shall be performed, and the most deviating value shall be excluded from the calculation of the average. If only one or two tests have been performed because of lack of undisturbed materials, this shall be noted and pointed in the report. For remoulded specimens the test shall be repeated until two successive tests yield identical values of cone penetration. [prCEN ISO/TS 17892-6:2003 (E)]

In the beginnings quite many types of cones were used, but after some investigations, it was decided to use four different standard cones, one with 400 gm in weight and 30° of apex angle, another with 100 gm in weight and 30° of apex angle, another with 60 gm in weight and 60° of apex angle, and a last one with 10 gm in weight and 60° of apex angle. It has to be noticed that the last cone will be specially used to determinate the liquid limit, and only in very soft clays for getting the undrained shear strength.

To estimate the resistance to the penetration of the cone it is necessary to have knowledge of the factors influencing the deformation of the clay. The resistance of the penetration of the cone depends not only on the modulus of shear but also on the viscosity of the clay. The influence of the viscosity depends upon several factors as for example the water content, the microscopic structure, and the rate of deformation of the clay. In the cone test the rate of deformation is different for different penetrations h but is always very high. The shear strength obtained will therefore be higher than in a slow shear test. [S. Hansbo 1957]

4.3.2 Study of the Cone Motion.

The motion of a body of mass m , subjected to a force vector P , is defined by

$$P = ma \quad (8)$$

where a is the acceleration vector of the body.

Using the engineering measurement system, the vertical motion of the cone may be written

$$P = \frac{Q}{g} z'' \quad (9)$$

where P is the vertical resultant of the forces acting upon the cone,

Q is the vertical weight of the cone,

g is the acceleration due to gravity, and

z is the depth of penetration at a certain time t $\left(z'' = \frac{d^2 z}{dt^2} \right)$.

Considering again the forces acting upon a cone element, Figure 39. Evidently the stresses τ and σ will vary along the cone surface and will depend not only on the failure stress τ_f , but also on the sensitivity and on the rate of shear.

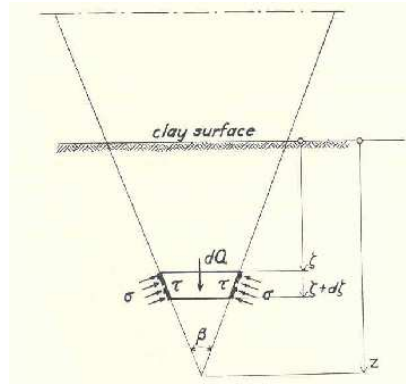


Figure 39. Forces acting upon a cone element during penetration. [S. Hansbo, 1957]

The exact expression looking at the Figure 6,

$$\begin{aligned} P &= Q - \cos \frac{\beta}{2} \iint \tau dA - \sin \frac{\beta}{2} \iint \sigma dA = \\ &= Q - 2\pi \tan \frac{\beta}{2} \int_0^z \tau(z - \zeta) d\zeta - 2\pi \tan^2 \frac{\beta}{2} \int_0^z \sigma(z - \zeta) d\zeta \end{aligned} \quad (10)$$

is therefore difficult of solution and is replaced by the approximate expression

$$P = Q - Tz^2 \quad (11)$$

where T is a function mainly of the shear strength τ_f of the clay and the cone angle β but is also influenced by the rate of deformation and by the sensitivity.

Equation (6) can thus be rewritten

$$z'' + gTz^2 / Q = g \quad (12)$$

whence

$$z' = \sqrt{C + 2gz - 2gTz^3 / 3Q} \quad (13)$$

The value of the constant of integration C is obtained from the boundary condition $z' = 0$ at $z = 0$ and is found to be zero.

If the final value of the depth of penetration is h , we have, since $z' = 0$ at $z = h$,

$$T = 3Q / h^2 \quad (14)$$

4.3.3 Relation between Shear Strength and Cone Penetration.

The investigation in 5.2.2 shows that it is possible to find an approximate relation, which is satisfactory for engineering purposes, between the undrained shear strength τ_f and the depth of penetration h . Thus, assuming $T = 3\tau_f / K$, Equation (10) becomes

$$\tau_f = KQ / h^2 \quad (15)$$

where K depends mainly on the cone angle β but also influenced by the rate of shear and by the sensitivity.

$$K = 0.85 \quad \text{for } \alpha = 30^\circ$$

and

$$K = 0.29 \quad \text{for } \alpha = 60^\circ$$

[M. Wood, 1990]

Obviously, Equation (11) would be of little interest if K varied widely for one and the same cone angle. In such a case no practical advantage would be gained over the previous interpretation of the fall-cone test. A comparison between Equation (11) and the results of the fall-cone test given in the final report of the Geotechnical Commission shows, however, that K is practically constant for each particular value of β as we can see in Figure 40, which is also confirmed by the following investigation.

Thus, the influence of normal variations in the rate of shear and the sensitivity appears to be small [S. Hansbo 1957]

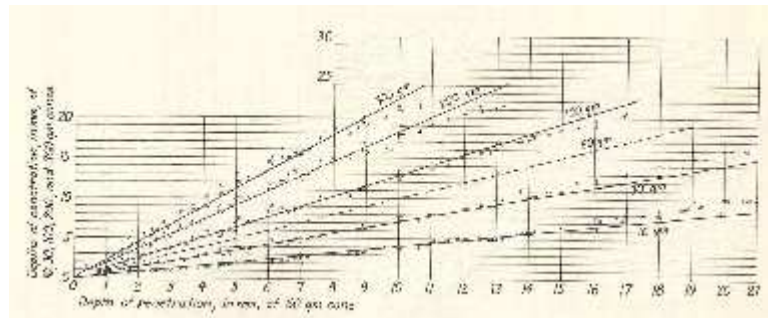


Figure 40. Results of cone test carried out by the Geotechnical Commission of the Swedish State Railways. The depths of penetration of 10 gm, 30 gm, 100 gm, 200 gm, and 300 gm cones represented as functions of the depth of penetration of the 60 gm cone according to Equation 11. Cone angle = 60° . [S. Hansbo, 1957]

This is specially a good test for soft soil and remoulded soil.

4.3.4 Test results.

Like in the unconfined compression test, in the three years project was planned to carry the fall-cone tests of the stabilized clays out as well. So, the samples we tested were the same as the ones we tested for the unconfined compression test.

4.4 Comparison of unconfined compression test and fall-cone test.

After carrying out lots of tests and observing the results we conclude that the Fall Cone Test is suitable for soft clays ($C_u < 100 \text{ kPa}$) and the Unconfined Compression Test is suitable for stiff clays and specially for stabilized soils.

5. SETTLEMENT PROPERTIES

5.1 Settlement and permeability.

Settlement is caused by application of increased stress on the soil. Before imposing a load on the soil, the soil has come to some sort of equilibrium, in which the soil particles can bear the weight of the overlying soil.

On applying an increased load to the soil through a footing, the soil is subjected to increased stress levels, which cause the particles to rearrange in a denser structure that can accommodate these higher stresses. The volume of voids between the soil particles decreases, and this decrease is reflected in settlement at the soil surface. For clays, the settlement is not instantaneous, because water travels slowly through a clay structure - voids are not continuous, and there are electrochemical attractions etc. There is some (small) component of elastic rearrangement; however, the bulk of the volume change occurs over weeks, months or years as the water is displaced from the more highly stressed areas. This rearrangement of particles is commonly called consolidation.

As we can see, there is a very important relationship between settlements and permeability. We will deal about this below.

5.2 The HUT model.

5.2.1 Introduction.

In the prediction of settlements for structures or embankments with or without vertical drainage a final settlement together with the settlement in the course of time is calculated. The most important parameter needed in the time-settlement calculation is the coefficient of consolidation c_v , conventionally determined from oedometer test results using e.g. Casagrande's or Taylor's curve fitting method.

$$c_v = \frac{k * M}{\gamma_w} \quad (16)$$

where k is the coefficient of permeability, M the deformation modulus and γ_w the unit weight of water. Terzaghi assumed in his theory that both the deformation modulus M and the coefficient of permeability k are constant at small stress differences. In the field settlement cases to which the consolidation theory is applied, the stresses often vary in a wide range. It is evident that k decreases with increasing stress as the volume of the pores network in the soil decreases and M increases with increasing stress in a homogeneous soil layer, but no conclusion can be readily drawn from the stress dependency of c_v by looking at the equation neither the test proves.

The tentative calculations carried out at the Laboratory of Soil Mechanics and Foundation Engineering of the Helsinki University of Technology showed that if the stress-dependence of the coefficient of consolidation c_v is taken into account, the calculation time for the total settlement will become two or three times longer (Ravaska and Vepsäläinen, 2001).

5.2.2 Permeability.

The relationship between permeability and strain can be mathematically modelled in many ways. Because seepage takes place in voids, it is natural that the void ratio e is most often used as strain variable. A comparison between the models developed until the moment was carried out at the Laboratory of Soil Mechanics and Foundation Engineering of the Helsinki University of Technology (HUT) by using data obtained from simultaneous oedometer test and permeability tests on soft clay samples. Equation 17 proved to be the best model for the compared clays, it is known as the HUT permeability model.

$$k = k_0 * \left(\frac{1+e}{1+e_0} \right)^\alpha \quad (17)$$

where k_0 permeability at zero strain and e_0 initial void ratio.

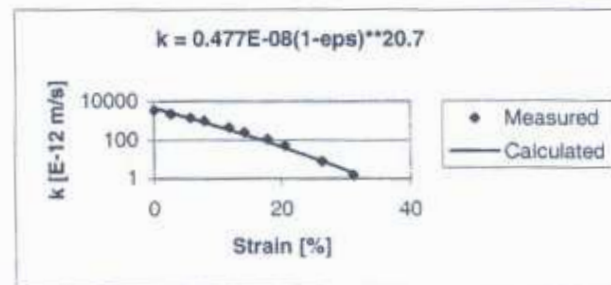
But actually, the void ratio is not very suitable to be used in practical computer programs, because it is not a universal material parameter. If we replace it by the strain ε , we will have many advantages. The HUT suggested relationship for the permeability and strain is as follows:

$$k = k_0 * (1 - \varepsilon)^\alpha \quad (18)$$

where ε is strain, k_0 permeability at zero strain and α constant. In this equation k_0 means the permeability of the sample at an initial state before loading, the constant α indicates the rate of change in permeability as the strain increases.

As an example we present Figure 41. Figure 41.a presents the modelling for bentonite for which permeability is in a logarithmic scale because of its wide range, and Figure 41.b for Don Valley clay with permeability in an arithmetic scale. Both samples were artificially sedimented and thus normally consolidated.

a)



b)

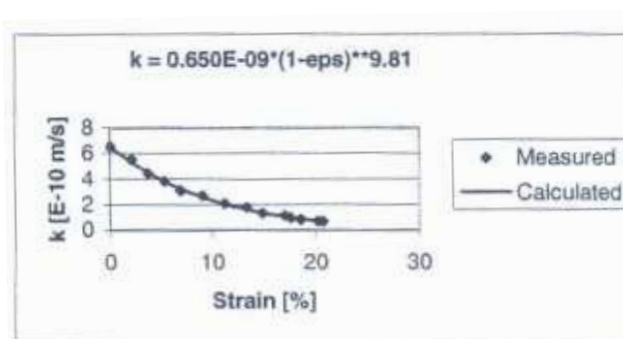


Figure 41.a) Bentonite. b) Don Valley clay. [O.T. Ravaska and P.E. Vepsäläinen. On the stress dependence of consolidation parameters, 2004]

5.2.3 Deformation modulus (M).

Clays in Nordic countries are mainly very young and soft due to their post-glacial origin. Their stress-strain behaviour also differs from that of most clays reported in the literature to which a linear relationship between the void ratio and stress in a semi-logarithmic scale can be applied, i.e. the use of C_c as a stress-strain parameter. A different solution was made in Scandinavia where C_c was abandoned and Ohde-Janbu model (Janbu 1979) was taken to the practical use. In Finland this took place in the 1970s. The Ohde-Janbu deformation model is as follows:

$$M = \frac{d\sigma'}{d\varepsilon} = m * \sigma_v * \left(\frac{\sigma'}{\sigma_v} \right)^{1-\beta} \quad (19.a)$$

$$\varepsilon = \frac{1}{m\beta} * \left[\left(\frac{\sigma'}{\sigma_v} \right)^\beta - \left(\frac{\sigma_0'}{\sigma_v} \right)^\beta \right] \quad \text{for } \beta \neq 0 \quad (19.b)$$

$$\varepsilon = \frac{1}{m} * \ln \frac{\sigma'}{\sigma_0'} \quad \text{for } \beta = 0 \quad (19.c)$$

where M is the deformation modulus (tangent modulus), m and β are the model parameters, σ_v is the reference stress of 100 kPa, $\sigma' = \sigma_0' + \Delta\sigma_z'$ the effective vertical stress during the consolidation process, σ_0' the initial effective vertical stress and $\Delta\sigma_z'$ the effective vertical stress caused by loading during consolidation.

Two parameters are needed for modelling all types of soils, β and m , they can be determined by a curve fitting method. The parameter m represents the compressibility of the soil and β the form of the stress-strain curve. If $\beta < 0$ (normally

consolidated clay), the curve is concave on a semi-logarithmic scale while if $\beta > 0$ (sand, overconsolidated clay), it is convex. In the special case of $\beta = 0$ (silt) it corresponds to the C_c model with a linear relationship between the logarithm of stress and the void ratio. Only in this case the relationship between m and C_c can be written as Equation 20.

$$m = \frac{(1 + e_0)}{C_c} \ln 10, \quad \text{when } \beta = 0 \quad (20)$$

where e_0 is the initial void ratio.

The numeric values of the parameters m and β can generally be characterized as the coarser material the higher are both values and therefore a certain correlation must exist between them, see e.g. Figure 42.

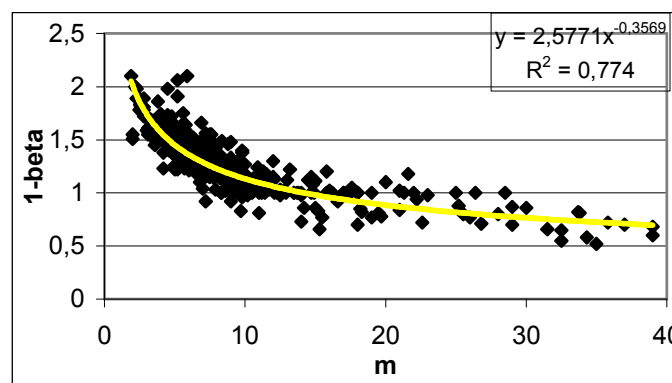


Figure 42. Correlation between m and $1-\beta$ for a number of Finnish clay. [A. Aalto, P. Vepsäläinen and O. Ravaska. Settlement calculation with stress-dependent parameters, 2005]

The stress dependence of the modulus number m was investigated by performing a large number of oedometer tests. Continuous samples were taken from deep and homogeneous clay formations in ten tests sites of the coastal part of Southern and Western Finland. At first glance the stress-dependence of modulus number m was not evident, but further data processing revealed the importance of water content of the samples in the case of stress-dependence. The dependence of modulus number m and preconsolidation pressure σ_c , can be seen in Figure 43 in which water content values (55%, 75%, 100% and 112%) form parallel limiting values of particular two-dimensional areas, Equation 21.

$$m = k_1 * \sigma_c + B \quad (21)$$

The slopes of the water content lines are constant as $k_1 = -0.08$ and the constant B varies between 8.8 and 14.5.

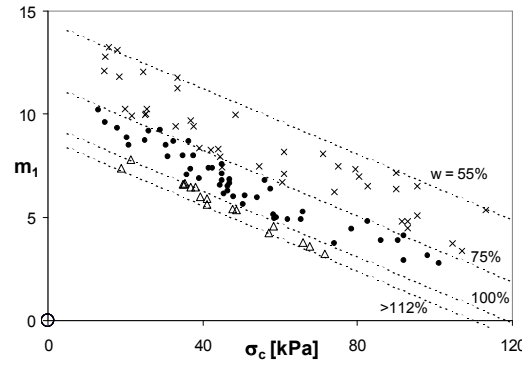


Figure 43. Modulus number m vs. Preconsolidation pressure σ_c in different water content areas. [A. Aalto, P. Vepsäläinen and O. Ravaska. Settlement calculation with stress-dependent parameters, 2005]

The function of constant B is parabolic between the water contents 55-112 %. For water contents more than 112 %, the function assumed to be constant as we can appreciate in Figure 44. Now the modulus number m gets the form of Equation 22 where the water content w is a dimensionless number.

$$m = -0.08 * \sigma_c + \frac{7.94}{w} \Rightarrow w \leq 55\% \quad (22.a)$$

$$m = -0.08 * \sigma_c + 10.6 * w^2 - 27.6 * w + 26.4 \Rightarrow 55\% < w < 112\% \quad (22.b)$$

$$m = -0.08 * \sigma_c \Rightarrow w \geq 112\% \quad (22.c)$$

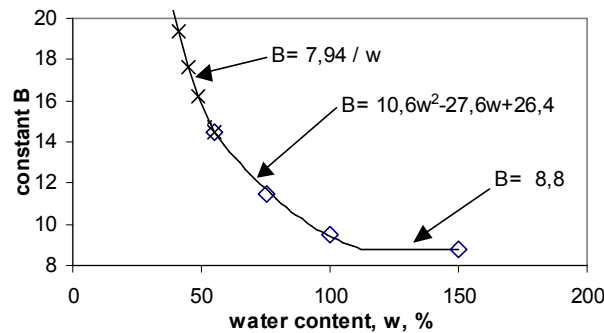


Figure 44. Water content w vs. Constant B . [A. Aalto, P. Vepsäläinen and O. Ravaska. Settlement calculation with stress-dependent parameters, 2005]

The correlation between the modulus number m and the stress exponent β was investigated using the same oedometer test data above. The correlation was quite moderate as we can see in Figure 45 and Equation 23.

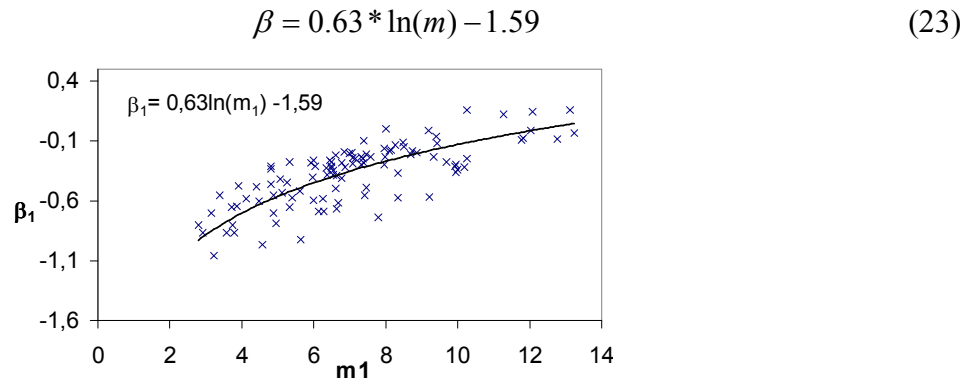


Figure 45. Modulus number m vs. stress exponent β . [A. Aalto, P. Vepsäläinen and O. Ravaska. Settlement calculation with stress-dependent parameters, 2005]

5.2.4 Coefficient of consolidation.

According to Terzaghi's consolidation theory the coefficient of consolidation has the form:

$$c_v = \frac{k * M}{\gamma_w} \quad (24)$$

where γ_w the unit weight of water. Substituting the Equations 18, 19.a to 16 gives:

$$c_v = \frac{k_0 * (1 - \varepsilon)^\alpha * m * \sigma_v}{\gamma_w} * \left(\frac{\sigma}{\sigma_v} \right)^{1-\beta} \quad (25)$$

At the same time the coefficient of consolidation is conventionally determined from the oedometer test results using Casagrande's and Taylor's method. Both methods are compared with the modelling presented above called the HUT method, the experience is told below.

5.2.5 Comparison between consolidation properties of natural and disturbed clay.

5.2.5.1 Introduction.

A soft clay sample from Hyvinkää, about 60 km north of Helsinki, was selected as an example of all the material tested. The sample was a block sample ($0.3 * 0.3 * 0.3 \text{ m}^3$) taken from a depth of 2.5-2.8 m. The soil type was fat clay including some silty layers.

Samples of three different types were tested. The first type was an undisturbed natural clay sample, the second a remoulded sample and the third one was a clay slurry produced by adding about 5% of extra water into the sample in order to have a complete disturbance.

5.2.5.2 Oedometer test.

Stepwise oedometer tests were performed on these samples, the stress-strain curves of which are presented in Figure 46.

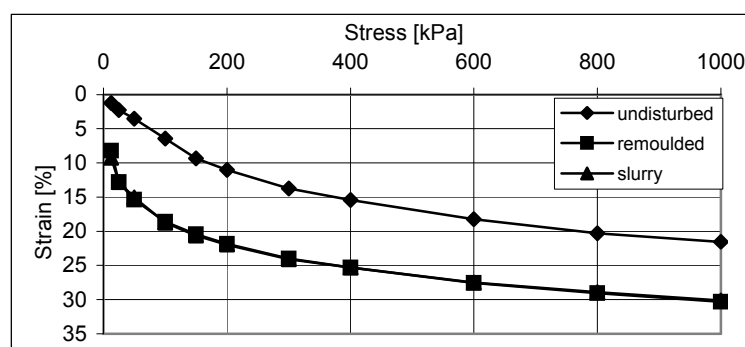


Figure 46. Stress-strain curves of the samples. [O.T. Ravaska, A. Aalto and M. Lojander. Consolidation properties of natural and disturbed clay, 2003]

From Figure 46 can be seen that the strain of a disturbed sample is bigger than that of an undisturbed one, which is natural because of a considerably high preconsolidation pressure of the undisturbed sample. It can also be seen that the

stress-strain curves of the disturbed samples are almost identical. At the beginning of the test the strain of the disturbed sample is fast and after the preconsolidation pressure it is parallel with the undisturbed sample.

5.2.5.3 Coefficient of consolidation.

The parameters of the Ohde-Janbu's model, Equation 19.a, together with the parameters of the C_c model for the tested samples are listed in Table 7. The disturbance caused a slight increase in both the modulus number m and the stress component.

Table 7. Consolidation parameters of the clay. [O.T. Ravaska, A. Aalto and M. Lojander. Consolidation properties of natural and disturbed clay, 2003]

Sample	e_0	M	β
Undisturbed	1.511	14.58	-0.015
Remoulded	1.454	21.17	0.075
Slurry	1.680	22.67	0.105

The coefficients of consolidation were determined at each loading step using Taylor's and Casagrande's methods and the HUT method according to Equation 25. The results are presented in Figure 47. The oscillation in the Taylor and Casagrande curves is due to the small loading steps, which makes the determination of the coefficient of consolidation inaccurate, particularly in Casagrande's method. The effect of the overconsolidation is clearly seen in Figure 47.a at the stress of 100 kPa. The c_v values according to Taylor and Casagrande are high compared to those at higher stresses. The same phenomenon is not seen in the HUT values, because the determination of c_v was made using values of m and β for a normally consolidated state.

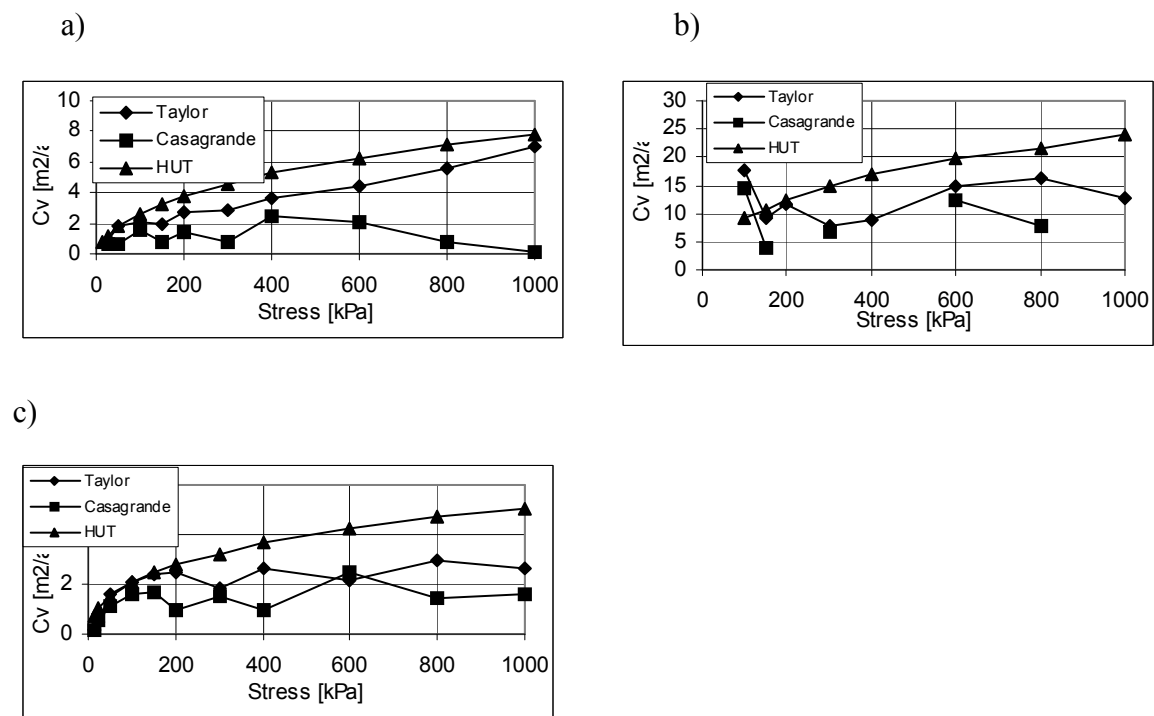


Figure 47. Coefficient of consolidation versus stress. a) undisturbed, b) remoulded, c) slurry. [O.T. Ravaska. Consolidation properties of natural and disturbed clay, 2003]

According to Leroueil, Magnan and Tavenas (1990), experience shows that:

$$c_v (\text{Casagrande}) < c_v (\text{Taylor}) < c_v = \frac{k * E'_{oed}}{\gamma_w} \quad (26)$$

where E'_{oed} is the secant modulus of the oedometer stress-strain curve corresponding to the tangent modulus M in Equation 19.a. Leroueil, Magnan and Tavenas (1990) present that c_v obtained by the method of Casagrande leads to underestimation of rates of settlement, but also that the values of c_v obtained using direct measurements of permeability k and compressibility E'_{oed} seem to be in closer agreement with reality.

The influence of disturbance in c_v is clearly seen in Figure 48, which presents the relationship between this and stress as measured by the HUT method. The curve of the undisturbed sample begins after the preconsolidation pressure has been exceeded, because the method cannot be used in overconsolidated soils with the same parameters. The coefficient of consolidation of the undisturbed sample is 2-3 times

that of the disturbed samples at the stress level after the overconsolidation pressure and the difference increases at higher stress levels. As the coefficient of consolidation is inversely proportional to the settlement time, it means that the settlement time in disturbed soils is much longer than in undisturbed soils.

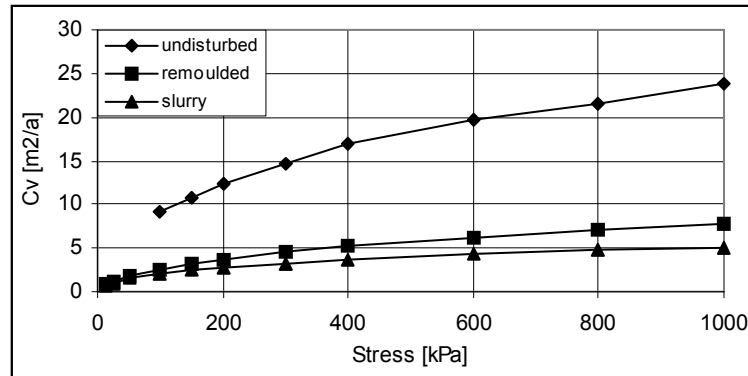


Figure 48. Coefficient of consolidation versus stress as determined by the HUT method. [O.T. Ravaska, A. Aalto and M. Lojander. Consolidation properties of natural and disturbed clay, 2003]

5.2.5.3 Conclusions.

From those laboratory tests we can conclude that:

- Strains of the disturbed samples were considerably bigger than those of the undisturbed sample, and strains of the disturbed (disturbed and slurry) samples were almost identical.
- Disturbance increased slightly both the modulus number m and the stress exponent β .
- Coefficient of consolidation values obtained support the order presented by Equation 26.
- Coefficient of consolidation of undisturbed sample was 2-3 times that of the disturbed samples at the stress level after the overconsolidation pressure, and the difference increases at higher stresses.

5.3 Laboratory settlement test with the oedometer test.

5.3.1 Regulations.

In order to know the settlement characteristics of the in our laboratory studied clays we carry out the oedometer tests by taking the CEN ISO/TS 17892-5:2003 code as reference. It is only a reference because this regulation is only applicable to undisturbed samples, what we are not obeying, we are working with stabilized samples.

5.3.2 Arrangement of the oedometric cell.

Firstly, the permeability of the porous stones we are going to use in the oedometer test shall be measured. For that we begin by cooking them, it means to saturate the stones by boiling them in distilled water, as we can see in Figure 49, for at least 20 minutes. After that they will be kept immersed in distilled water until being used.

The process to get the permeability of the porous stones is the same as the one to achieve the permeability of the clay inside the odometer ring, so we are not going to explain now the method to get the stone value, it will be done afterwards when we tell the achievement of clay permeability.



Figure 49. Porous stones inside the boiling water.

The porous stones role is to allow free drainage of water, while preventing intrusion of soil particles into their pores. The material must be corrosion-resistant and non-compressible under the maximum stress likely to be applied during the test. They shall be thick enough to prevent breakage under load.

Secondly, a rubber ring inside the hole of the oedometric cell shall be placed, this avoids water flowing through it. We can appreciate that in the Figure 50.

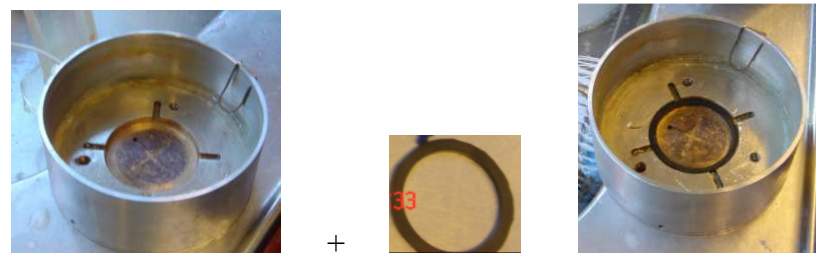


Figure 50. We put the rubber ring inside the oedometric cell.

Third step lies in fitting the group of layers we can see in Figure 51 inside the cell.

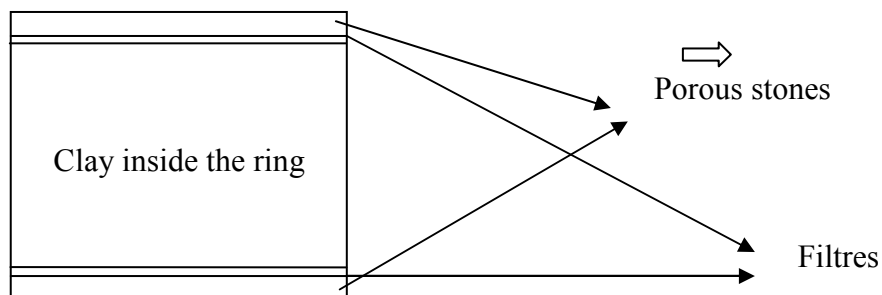


Figure 51. Layers we arrange into the oedometric cell.

Filter paper is used to prevent intrusion of the soil into the porous stones. However, the permeability of the stones and the filter paper shall be sufficiently high to prevent retardation of the drainage of the specimen.

The diameter of the top porous plate shall be about 0.5 mm less than the internal diameter of the oedometer ring, and may be tapered toward the upper face to

minimize the risk of binding due to tilt. The bottom porous disc shall be similar to the top one, but tapered towards the lower face. [CEN ISO/TS 17892-5:2003]

The oedometer cell shall accept the oedometer ring with a push fit and shall be rigid enough to prevent significant lateral deformation of the ring when under load.

Next, the instrument in Figure 52 shall be screwed down in the cell, and after that the loading cap shall be left as we can see in Figure 53. As the porous stones, the loading cap shall be about 0.5 mm less than the internal diameter of the oedometer ring, and shall be rigid enough to ensure negligible deformation under load. The regulation in use tells that “If porous stones with a thickness of less than 6 mm are used, then the loading cap shall have perforations or grooves to allow the free drainage of pore water”, so that is the reason why our top element is perforated as we observe in Figure 53.



Figure 52. a) Instrument to screw down on the cell. b) Screwing the instrument.



Figure 53. a) Loading cap, b) Cell before leaving on the oedometer machine, c) Cell on the oedometer machine.

The final view of the oedometer cell is the one we can appreciate in Figure 54.

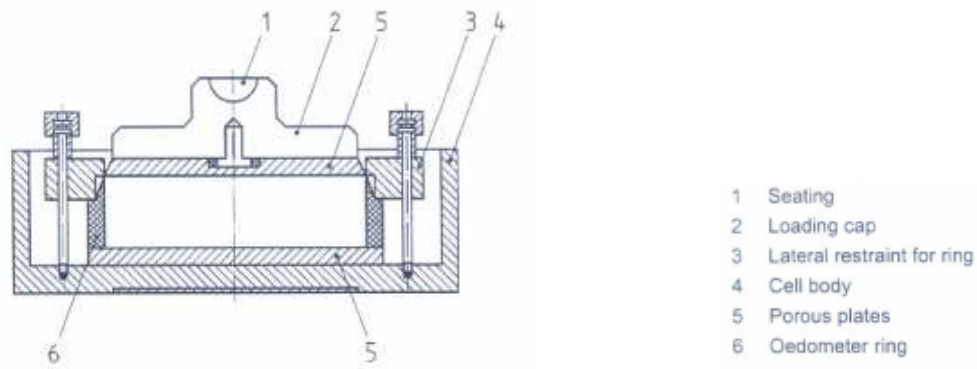


Figure 54. General arrangements of typical oedometer cells. [CEN ISO/TS 17892-5:2003]

All components shall be made of materials which are not corrodible by electro- chemical reaction with each other, or the soils and the pore water.

5.3.3 Environment.

Tests specimens shall be prepared in an environment which avoids significant loss or gain of soil water. If the preparation process is interrupted the specimen shall be protected by wrapping in plastic sheet or clingfilm. [CEN ISO/TS 17892-5:2003]

The apparatus shall be protected against sunlight, local sources of heat and draughts. [CEN ISO/TS 17892-5:2003]

5.3.4 Deformation gaugement.

The deformation gauge we use in our laboratory is the one we can observe in Figure 55, it obeys the code. It measures the vertical displacements of the samples.



Figure 55. Vertical displacement measurer.

5.3.4 Loading.

The loading should be started immediately after the specimen has been prepared but, if a short delay is unavailable, the specimen shall be protected by wrapping in thin plastic or clingfilm until ready for testing.

Once the cell has been left on the oedometer apparatus, a small seating pressure shall be applied to the specimen not exceeding 3 kPa including the weight of the top cap and porous plate. The deformation gauge shall be secured in position and the initial reading corresponding to zero deformation shall be recorded. The timer shall be set to zero.

The code sets the initial vertical stress to be applied on the samples. It depends on the type of soil; Table 8 suggests some initial values.

Table 8. Suggested initial pressure. [CEN ISO/TS 17892-5:2003]

Soil Consistency	Initial pressure
Stiff	Equal to σ'_0 or the next higher recommended pressure above σ'_0 if σ'_0 is less than σ'_s ⁽¹⁾ .
Firm	Somewhat less than σ'_0 , preferably using the next lower recommended pressure.
Soft	Appreciably less than σ'_0 , usually 25 kPa or less.
Very soft	Very low, typically 6 kPa or 12 kPa. Initial consolidation under small load will give added strength to prevent squeezing out under next load increment.

(1) Swelling pressure, i.e. the pressure required to maintain constant volume (i.e. to prevent swelling) when a soil is flooded with water.

We are working on the very soft conditions, so the initial pressure to apply the sample is 6 kPa, when testing soft soils the initial stress should be restricted in order to avoid yielding.

In the normal procedure, the one we make use of, each stress shall be double the previous stress (increment ratio = 1). A suggested stress sequence is: 6, 12, 25, 50, 100, 200, 400, 800, 1600 kPa, and the following reading intervals are proposed: 6'',

15'', 30'', 1', 2', 4', 8', 15', 30', 1h, 2h, 4h, 8h, 24h. A period of 24 h is normal, it is supposed the consolidation finished within this duration. In our case we do not follow strictly the proposed intervals of time, but our samples are tested for 24 hours.

If the apparent preconsolidation pressure σ'_p was to be determined, the load increment ratio would be decreased near the expected value, what may affect the time-settlement plots and make interpretation of c_v more difficult. [CEN ISO/TS 17892-5:2003]

When an extra load is going to be applied on the sample, the deformation gauge reading shall be recorded as the initial reading for the load increment stage. The required load shall be carefully applied, without jolting, within a period of 2 s. At the same instant the timer shall be started and the small seating load shall be removed. We can see how the loads are sequentially arranged on the back side of the oedometer apparatus in Figure 56.



Figure 56. Loads on the plates of the back side of the oedometer apparatus.

The consolidation cell shall be filled with water to the top of the upper porous plate as we can see in Figure 57. If the specimen begins to swell this shall be prevented either by applying the next higher vertical stress in the sequence and re-starting the timer or by determining the swelling pressure.

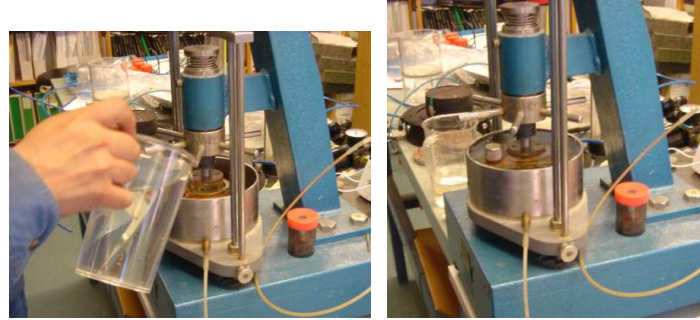


Figure 57. a) pouring the water into the cell, b) full-filled cell of water.

As a general view of the apparatus during the loading we can see the Figure 58.



Figure 58. General view of the oedometer apparatus.

5.3.5 Unloading.

The unloading portion of the logarithm of pressure/void ratio curve is sometimes required, but taking readings during unloading is optional. Normally the number of unloading stages should be at least two, however, more may be used to provide curves with reasonably equally spaced points on the plotted graph e_0 vs. σ'_v .

The deformation gauge reading shall be recorded at the end of the last loading stage. This becomes the initial reading for the decrement stage.

The vertical stress shall be reduced on the specimen carefully to the selected value. At the same instant the clock shall be started.

Deformation gauge readings shall be recorded at suitable intervals of time such as those given for the loading episode, until swelling is virtually completed.

The same procedure for next loading and unloading cycles shall be followed, if required.

5.3.6 Dismantling the apparatus.

When the equilibrium under the final vertical stress is indicated, the water shall be drained from the cell and about 15 min shall be allowed for free water to drain from the porous plates. Any excess water shall be removed from within the cell with an absorbent tissue.

Afterwards the vertical stress shall be removed from the specimen, the cell shall be removed and dismantled.

Finally, the density and water content shall be determined from the whole specimen. Any soil adhering to the porous plates to the filter papers shall be included.
[CEN ISO/TS 17892-5:2003]

5.4 Oedometer test results.

As result of the oedometer test we get a group of time values (t) and their respective height of the sample (H) for a particular applying load (σ_1). With those results we can draw two very useful graphs in order to get very important properties of the soils.

a) Height vs. time graph in order to achieve the c_v value.

Terzaghi's equation can be written in non-dimensional terms and after calculations to get the Equation 26 as solution.

$$\bar{U} = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \cdot \exp(-M^2 \cdot T_v) \quad (27)$$

where \bar{U} is the mean value of U_v (proportion of the pore pressure dissipated after time t and also the proportion of the total volume change which has taken place at that time), T_v is called the time factor and $M = \frac{1}{2}\pi(2m+1)$ where $m=1,2,3,\dots$

he time factor can be expressed as:

$$T_v = \frac{c_v \cdot t}{d^2} \quad (28)$$

where d is the length of the maximum drainage path, t is the time in the consolidation process and c_v is the coefficient of consolidation.

From these results the coefficient of consolidation can be calculated with different methods. These methods calculate c_v from the oedometer results as we said above.

The methods most used are Taylor's method and Casagrande's method that use the chart we are talking about.

Taylor Method.

For 90% consolidation ($\bar{U}=0.9$), assuming uniform distribution of exceed pore water pressure with depth, T_v (time factor) = 0.848 from the Terzaghi time factor values. Now:

$$T_v = \frac{c_v \cdot t}{d^2} \Rightarrow c_v = \frac{T_v \cdot d^2}{t} \quad (29)$$

$$\text{Therefore if } U_v = 90\% \text{ then } c_v = \frac{0.848 \cdot d^2}{t_{90}} \quad (30)$$

t_{90} is found from Figure 59, the one we can get from the oedometer test and d = drainage path length appropriate to the laboratory test: with “one-way” drainage d = sample thickness; with “two-ways” drainage d = (sample thickness)/2.

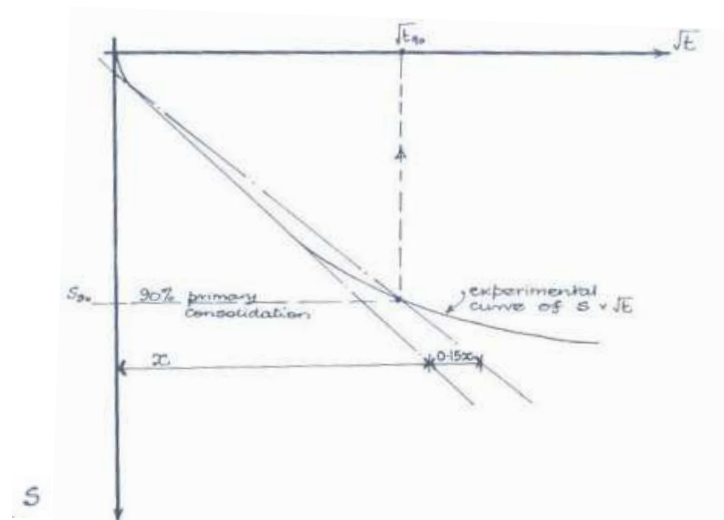


Figure 59. Prediction of settlement rate: Taylor's method. [B. Vickers, 1978]

During the three years project we make use of this method to get the coefficient of consolidation in our laboratory. In Figure 60 we can observe how we do it by utilizing excel program.

In this case it deals with a sample of clay from Vanttila quite near to Helsinki. Its characteristics appear in Table 9 and the applying load is $\sigma_1 = 5.5$ kPa.

Table 9. Characteristics of the sample and the applying load.

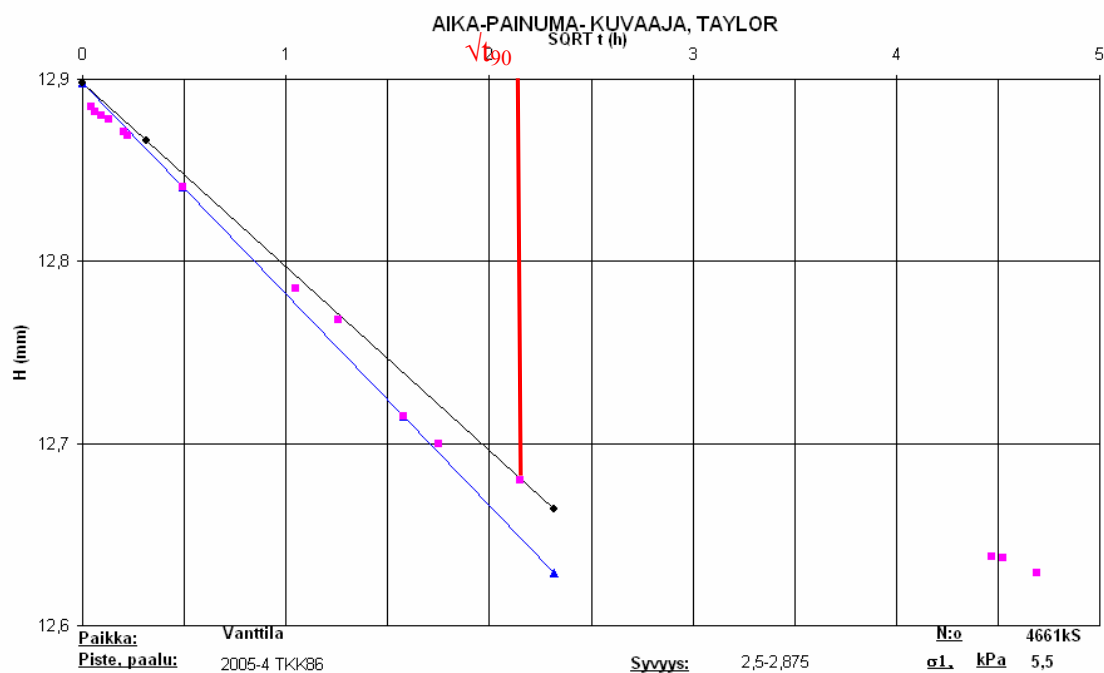
Oedometer								
Characteristics.	11k	h_0 , cm	1.67	A, cm ²	50.26	V, cm ³	83.7	σ_1 , kPa
								5.5

The results from the oedometer test are the following.

Table 10. Results of the oedometer test.

TIME			TIME	HEIGHT
h	min	s	t (h)	H (mm)
		6	0,00166667	12,885
		15	0,00416667	12,882
		30	0,00833333	12,88
	1		0,01666667	12,878
	2,5		0,04166667	12,871
	3		0,05	12,869
	14,5		0,24166667	12,841
1	6		1,1	12,785
1	35		1,58333333	12,768
2	29		2,48333333	12,715
3	4		3,06666667	12,7
4	37		4,61666667	12,68
20			20	12,638
20	30		20,5	12,637
22			22	12,629

So turning the “time” into the “square root of time” we can get the chart in which we can read t_{90} .

Figure 60. Getting t_{90} with Taylor's method.

Once we got the t_{90} value we can achieve the c_v :

$$c_v = \frac{0.848 * d^2}{t_{90}} = \frac{0.848 * H_{50}^2}{t_{90}} = \frac{0.848 * (12.78/20)^2}{2.16^2 * 3600} = 2.06E-5 \text{ cm}^2 / \text{s}$$

Casagrande method.

In this case, observations of settlement and time are plotted as settlement versus the logarithm of time and the characteristic shape shown in Figure 60 is produced.

From the tabulated Terzaghi U_v/T_v relationships, at 50% consolidation ($U_v = 50\%$) the corresponding time factor $T_v = 0.197$, assuming initial uniform distribution of excess pore-water pressure with depth.

$$T_v = \frac{c_v * t}{d^2} \quad (31)$$

and also

$$T_v = 0.197 = \frac{c_v * t_{50}}{d^2} \quad (32)$$

therefore

$$c_v = \frac{0.197 * d^2}{t_{50}} \quad (33)$$

t_{50} is determined by the construction shown in Figure 61 and d is the appropriate drainage path length.

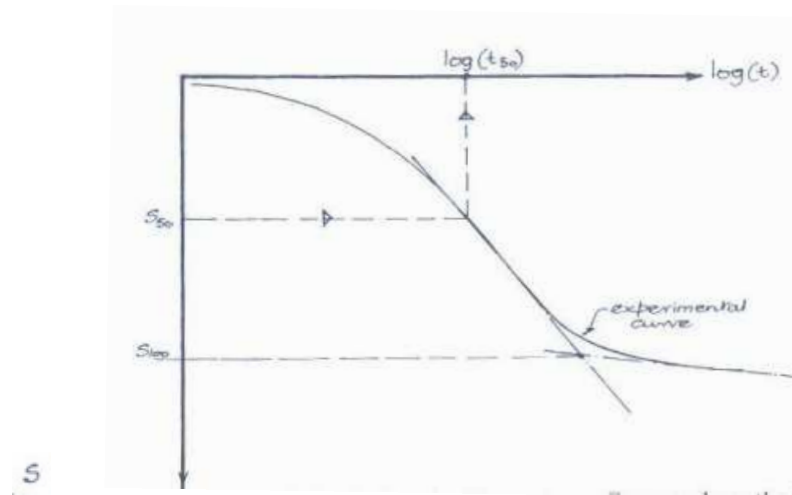


Figure 61. Prediction of settlement rate: Casagrande Method. [B. Vickers, 1978]

By using the same tests as the one we used before, we can turn the “time” variable into the “logarithm of time” and achieve the t_{50} value, which allows to apply Casagrande’s method.

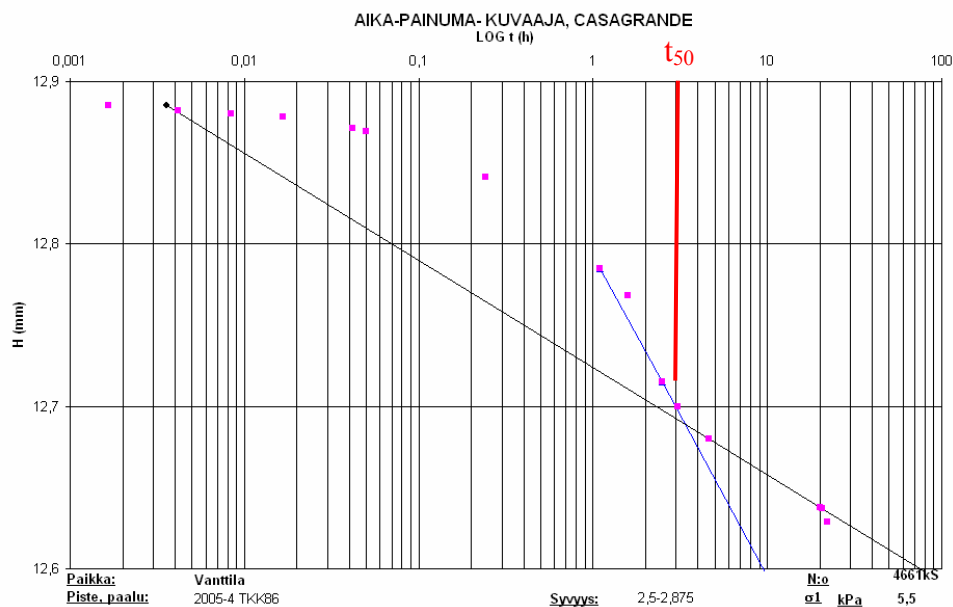


Figure 62. Getting t_{50} with Casagrande’s method.

Once we got the t_{50} value we can achieve the c_v :

$$c_v = \frac{0.197 * d^2}{t_{50}} = \frac{0.197 * H_{50}^2}{t_{50}} = \frac{0.197 * (12.79 / 20)^2}{1.07 * 3600} = 2.08E - 5 \text{ cm}^2 / \text{s}$$

As we can see by comparing Taylor's and Casagrande's methods, the c_v results are very similar.

b) 1D strain vs. stress graph.

In this case we are going to work with another sample whose name is 4709KSSW. It is from Otaniemi, Espoo. Its characteristics appear in Table 11.

Table 11. Characteristics of the sample.

Place:	HUT							N:o	4709kssw
Piston:	2006-1 Norj 3				Depth:	1,79-1,89			
Oedometer									
Oed. no.	9k	h ₀ , cm	2,0	A, cm ²	20	V, cm ³	39,7	σ ₁ , kPa	
Remarks	Cured underwater in oedometer 9days								

By carrying out the oedometer test we achieve the following results.

Table 12. Results from the oedometer test.

σ_1 (kPa)	h_1 (mm)	ε_1 %	$v = e+1$	$\ln \sigma_1$
6,25	19,852	0,02	2,864	1,83
12,5	19,841	0,08	2,862	2,53
25	19,819	0,19	2,859	3,22
50	19,780	0,38	2,853	3,91
100	19,731	0,63	2,846	4,61
200	19,479	1,90	2,810	5,30
400	19,199	3,31	2,770	5,99
800	18,690	5,87	2,696	6,68
1200	18,055	9,07	2,605	7,09
25	18,325	7,71	2,644	3,22
50	18,302	7,83	2,640	3,91
100	18,268	8,00	2,635	4,61
200	18,216	8,26	2,628	5,30
400	18,152	8,58	2,619	5,99
800	18,088	8,9	2,609	6,68

We can see in red colour the data we get from the oedometer test and in black colour values we calculate from the data in red.

If we draw the $\varepsilon_1(\%)$ values against the σ_1 values in a logarithmic scale we achieve the Figure 63, for the sample we are examining with. We can appreciate as well the associated parameters.

Place:	HUT			N:o	4709kssw
Piston:	2006-1 Norj 3			Depth:	1,79-1,89
σ_{v0} (kPa)=	0	$\beta_1 = 0,730$	$\beta_2 = 0,595$		
σ_P (kPa)=	100	$m_1 = 86,35$	$m_2 = 328,7$		

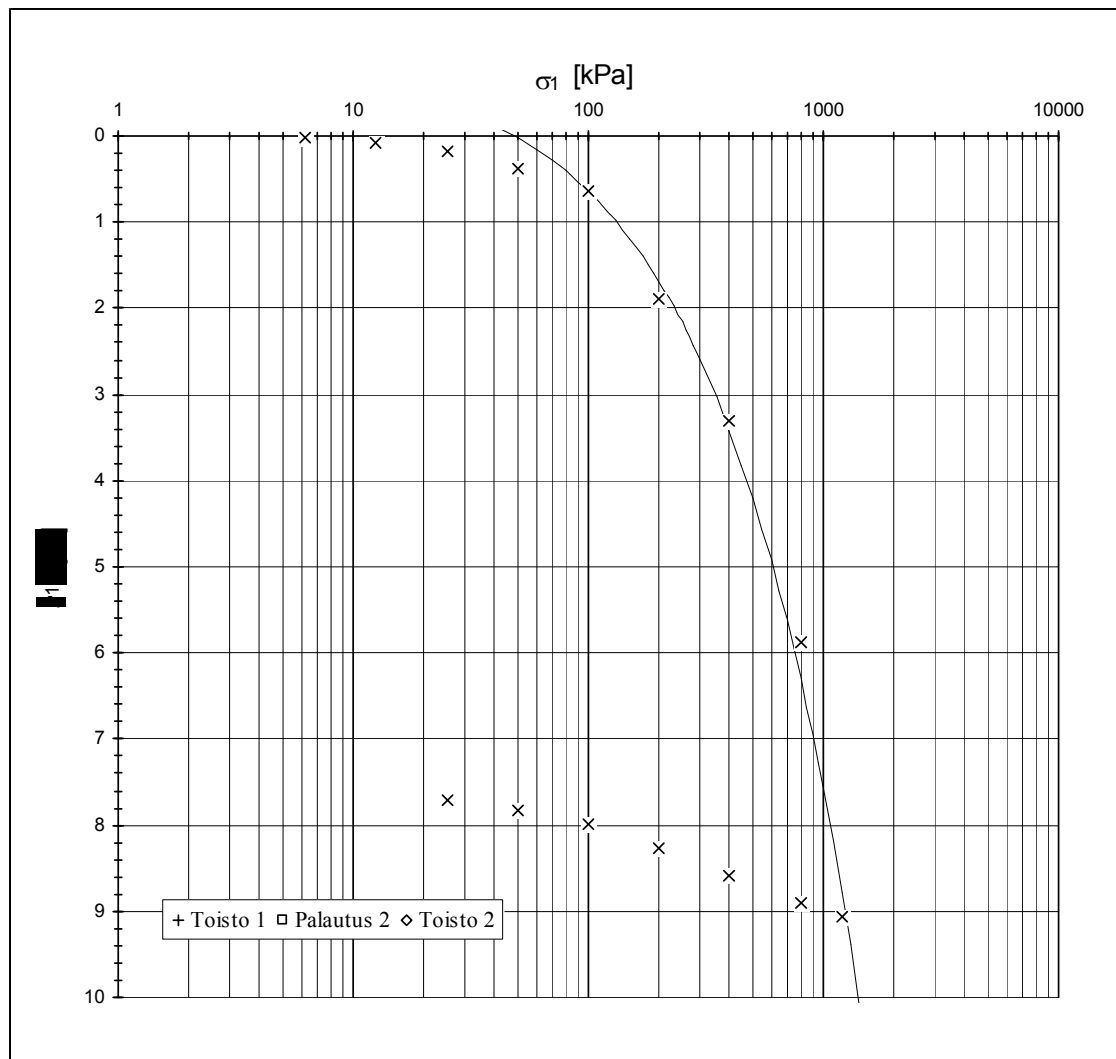


Figure 63. One dimensional Stress- Strain graph of the oedometer test for the 4709kssw sample.

6. PERMEABILITY

6.1 Modelling permeability.

6.1.1 Introduction.

The relationship between permeability and index properties of soil has interested researchers from the beginning of modern geotechnics. It is evident that permeability of soils decreases during consolidation, what is the most evident erroneous assumption in Terzaghy's consolidation theory. In settlement calculations it can be easily taken into account by modelling it mathematically using a simple model.

Over the last few decades the modelling of soil and the use of CAD have become more commonly used among geotechnical engineers and this has led to an increasing need for mathematical permeability models that can be incorporated into the design programs.

Because seepage takes place in the voids, it is natural that the void ratio e is most often used as a strain variable when modelling permeability in consolidation.

6.1.2 Models.

Taylor's method (1948)

Taylor's model fits the permeability with an almost linear relationship between the void ratio to the void ratio function and the permeability. For sand the relationship is presented by:

$$k = C \cdot \frac{e^3}{1 + e} \quad (34)$$

where C may be a constant or may vary slightly with the change in void ratio though in this study it will be considered as a constant. For fine grained soils the relationship is:

$$e = C \cdot \log k \quad (35)$$

Raymond's method (1966)

The previous logarithmic form was modified by Raymond in 1966. He related the permeability and the void ratio by the next formula.

$$e = I \cdot \log(k / k_n) \quad (36)$$

where I is a constant and k_n the apparent value of k at zero void ratio e_0 .

Rowe and Li's method (2002)

They show a relationship between the permeability and the void ratio with the next exponential form:

$$k = k_0 \cdot \exp\left(\frac{e - e_0}{C_k}\right) \quad (37)$$

where k_0 is the reference permeability and e_0 the initial void ratio.

Samarasinghe et al. (1982)

Because experimental results do not always follow a linear relationship, Samarasinghe et al (1982) changed the equation (34) into a more common form replacing the constant exponent 3 by a variable n :

$$k = C \cdot \frac{e^n}{1+e} \quad (38)$$

where C is a constant in the same unit as k , n is another constant depending on the type of soil including also fine-grained soils. Both C and n are positive. The variable e has two positions in the model function and the parameters, if exact values are desired, have to be determined using a procedure of trial and error.

Kulklik et al. (1998)

Kulklik et al. (1998) presented the relationship of k and e in a power law form, which also includes the unsaturated flow:

$$k = k_{e0} \cdot \left(\frac{e}{e_0} \right)^{m_1+m_2} \cdot \left(\frac{1}{S_{r0}} \right)^{m_2} \quad (39)$$

where k_{e0} is permeability corresponding to e_0 and m_1 , m_2 are material parameters and S_{r0} initial saturation. For saturated soils it can be written in the form:

$$k = k_{e0} \cdot \left(\frac{e}{e_0} \right)^\beta \quad (40)$$

where $\beta = m_1 + m_2$.

Fox and Baxter (1997)

A model in which the variable is the stress instead of the void ratio, published by Fox and Baster (1997), was based on the assumption that the logarithm of vertical permeability varies linearly with the logarithm of effective stress in consolidation:

$$k = k_c \cdot \left(\frac{\sigma}{\sigma_c} \right)^{-A} \quad (41)$$

where k_c is the vertical permeability corresponding to an arbitrary stress σ_c . The same relationship was later used for overconsolidated clay by **Kodikara and Rahman (2002)** with $A = C_r/C_k$. C_k and C_r are constants.

HUT model (2002)

At the beginning of the year 2002, a project aiming at improvement of settlement calculations was started at the Soil Mechanics and Foundation Engineering Laboratory of the Helsinki University of Technology (HUT). One of the sub-tasks of the project concerned permeability of fine-grained soils during consolidation and it should result in a permeability model well adaptable to the computer program used to calculate settlement.

Interpretation routines at HUT had conventionally included permeability modelling using a logarithmic model:

$$\varepsilon = A \cdot \ln k + B \quad (42)$$

where A , B are model parameters, ε is vertical strain and k the permeability.

Basically it is in the same form as presented in Equations 35, 36 and 37, and matches fairly well the permeability of clay as we can see in Figure 64.

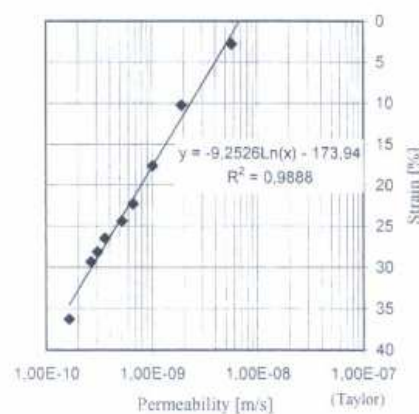


Figure 64. Natural logarithm based modelling of permeability of Murro 3002k clay. Permeabilities calculated according to Taylor. [O. Ravaska & A. Aalto. Modelling permeability in consolidation, 2003]

A need for a new permeability model arose when considering the stress dependence of the coefficient of consolidation c_v which is linked to the permeability of fine grained soils by Terzaghi's consolidation theory. The intention was to model the relationship between the stress and the coefficient of consolidation so that this relationship could be given to the computer program by means of input parameters only. Because the strain ε was inevitably needed for others purposes contrary to the void ratio e , it was selected as the deformation parameter in order to minimize the number of parameters the model and accordingly input parameters in the computer program. The selected model (Ravaska & Vepsäläinen, 2001) is:

$$k = k_0 \cdot (1 - \varepsilon)^\alpha \quad (43)$$

where k is permeability at the strain ε , k_0 permeability at zero strain and α a constant.

The model has several advantages:

- deformation is presented in the form of the strain ε , a parameter which is needed for other purposes in most cases.
- k_0 has a distinct physical meaning, i.e. permeability in a natural state of a sample before loading, unlike the corresponding constants in other models.
- the model is valid as well for normally consolidated soil as overconsolidated soil unlike models in which the stress is parameter.
- the power law for enables determination of the model parameters easily by using common program as Microsoft Excel available for anybody.

6.1.3 Comparison of models.

6.1.3.1 Modelled clays.

A comparison of models program was carried out in the Laboratory of Soil Mechanics and Foundations Engineering of the Helsinki University of Technology since 2001. A number of different undisturbed clay samples from the test sites of Taasia, Murro, Vanttila, Haarajoki, Otaniemi and Parainen were tested by using oedometer. Samples were taken from different depths. One sample was remoulded before testing and another one was stabilized with a small amount of cement. Oedometer tests were performed on each sample with constant loading steps in addition to the conventional way of doubling the load at each step.

Table 13. Sample information. The first letter in the sample code means the sampling site (T = Taasia, M = Murro, V = Vanttila, H = Haarajoki, O = Otaniemi, P = Parainen). [O. Ravaska & A. Aalto. Modelling permeability in consolidation, 2003]

Sample	Depth M	Clay content %	Water content %	Void ratio e_0
T2661	4.6		91.3	2.494
M2982	7.5	29	78.5	2.188
M2983	4.5	34	92.9	2.550
M3001	3.1	34	89.1	2.480
M3002	3.1	34	90.4	2.538
M3003	4.1	34	93.2	2.521
M3345	*	35	94.1	2.509
M3407	10.2	30	66.6	1.780
M3416	19.2	39	60.9	1.664
M3424	13.1	33	67.2	1.865
M3477	5.8	28	88.7	2.388
V3369	3.0	48	123.5	3.410
H3314	21.6	28	60.6	1.630
H3315	21.5	28	69.0	1.860
O3343	2.1	80	69.4	1.930
P3230	**	76	96.6	2.749

* remoulded
** stabilized with cement

6.1.3.2 Permeability tests.

Modelling permeability in consolidation provides that permeability tests are performed with consolidation tests. In these tests the method we worked with was the “Permeability testing in the triaxial cell” method that will be explained in the next section.

6.1.3.3 Modelling.

The suitability of Equations 40, 41, 42 and 43 in modelling the permeability test data was compared using Microsoft Excel. Also some successful fittings, with good correlation factors, were made using the model presented by Equation 38. This model was not, however, included in the comparison because of the difficulty of determining the model parameters. An example of the fittings is presented in Figure 65.

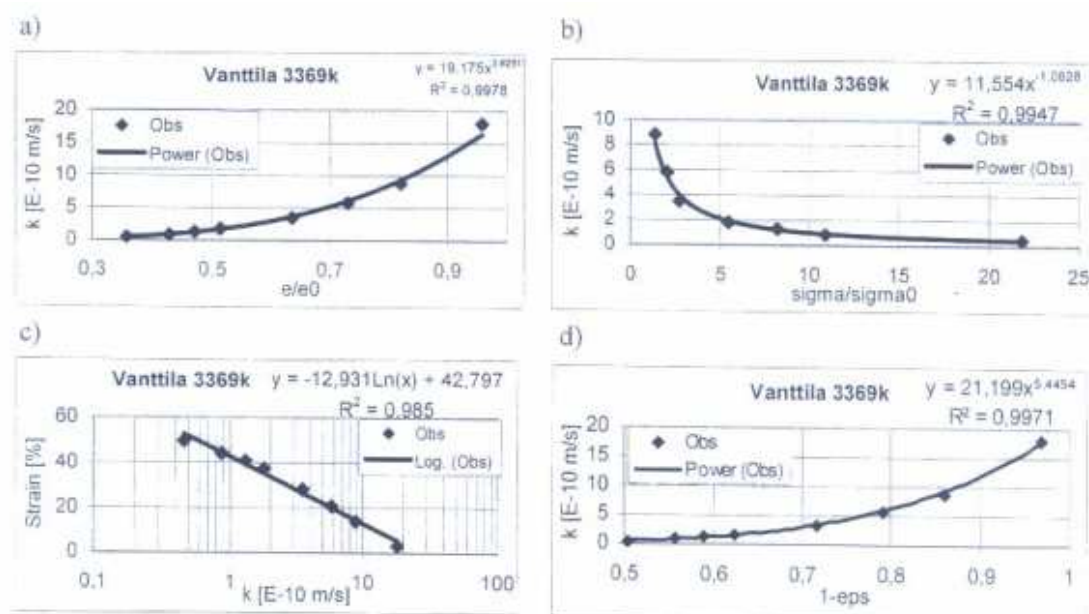


Figure 65. Clay sample V3369 as modelled by logarithmic and power function models. a) Eq. 40, b) Eq. 41, c) Eq. 42, d) Eq.43. [O. Ravaska & A. Aalto. Modelling permeability in consolidation, 2003]

The correlation factors obtained in the curve fittings are presented in Table 14. The table shows that the logarithmic k model (42) gave the greatest number of highest correlation factors and the e powerlaw model (40) as many, but they both also included five lowest values. The σ model (41) gave the highest value of all fittings, i.e. 0.9999 for the sample M3001, but also some very low correlations.

Table 14. Comparison of models using correlation factor R2 values from curve fitting.
[O. Ravaska & A. Aalto. Modelling permeability in consolidation, 2003]

Sample (no tests)	$k = k_{e0} \cdot (e/e_0)^B$ (7)	$k = k_e \cdot (\sigma/\sigma_e)^{-A}$ (8)	$\varepsilon = A \cdot \ln(k) + B$ (9)	$k = k_e \cdot (1-\varepsilon)^a$ (10)
T2661 (3)	0.9690*	0.9845	0.9886	0.9769
M2982 (4)	0.9947*	0.9976	0.9972	0.9967
M2983 (4)	0.9938*	0.9956	0.9966	0.9961
M3001 (4)	0.9995	0.9999	0.9885*	0.9969
M3002 (9)	0.9471	0.9320*	0.9757	0.9762
M3003 (5)	0.9647*	0.9876	0.9887	0.9753
M3345 (7)	0.9951	0.9771*	0.9785	0.9904
M3407 (4)	0.9805	0.9670*	0.9939	0.9880
M3416 (11)	0.9765	0.9825	0.9543*	0.9676
M3424 (11)	0.9639*	0.9726	0.9832	0.9720
M3477 (7)	0.9871	0.9667*	0.9713	0.9823
V3369 (8)	0.9978	0.9947	0.9850*	0.9971
H3314 (5)	0.9506	0.9408*	0.9695	0.9824
H3315 (7)	0.9757	0.9694	0.9572*	0.9607
O3343 (5)	0.9570	0.9548*	0.9668	0.9598
P3230 (5)	0.9800	0.7770	0.9702*	0.9772
Average	0.9771	0.9621*	0.9791	0.9810

bold: highest value for the sample

* lowest value for the sample

highest average value indication that it can be regarded as the most reliable model of those in this compared.

6.1.3.4 Conclusions.

Four permeability models from the literature together with the HUT model were compared. Assessment of the model equations:

- models (42) and (43) require only two input parameters, the others three,
- multipliers only in models (40) and (43) have a physical meaning.
- the multiplier in model (38) is difficult to evaluate,
- model (41) requires different parameters for normally and overconsolidated states,

The accuracy of the models (40), (41), (41) and (43) was tested on 16 clay sample data.

The results can be summarized as follows:

- models (40) and (42) gave both good and poor correlations,
- model (41) gave good, poor and very poor correlations,

- model (43) gave no poor correlation and best correlation on average.

As result of the comparison, Equation 43 can be regarded as the most reliable model with only two parameters, which can be easily determined. Additionally, the multiplier has a distinct physical meaning.

6.2 Testing.

6.2.1 Measurement methods.

The permeability of saturated soils can be measured in either the laboratory or the field.

Because of the very large range of permeability in soils, no single method is suitable for all cases. The types of equipment most generally used are:

- the constant head permeameter, for permeabilities above 10^{-4} m/s,
- the falling head permeameter, for values of k between 10^{-4} and 10^{-7} m/s
- ,
- the method of permeability testing in the triaxial cell,
- the method of permeability testing in the oedometer apparatus, and
- the Rowe's apparatus method.

Next, we describe accurately all of them.

The constant head permeameter

The soil sample is confined within a Perpex cylinder. A wire mesh filter is fitted at the upper and lower ends of the sample to prevent fine material being washed out. Before starting the test a vacuum can be applied to the sample in an attempt to perform the permeability test at a degree of saturation approaching 100%.

A constant-head supply is connected to the top of the sample and seepage takes place down through the soil. Once equilibrium has been reached, the rate of flow is measured, it means that the volume of water passing through the sample in a unit of time is recorded. The permeability will be:

$$k = \frac{q}{A \cdot i} \quad (44)$$

where q is the rate of flow, A is the cross section of the sample and i is the value of the hydraulic gradient.

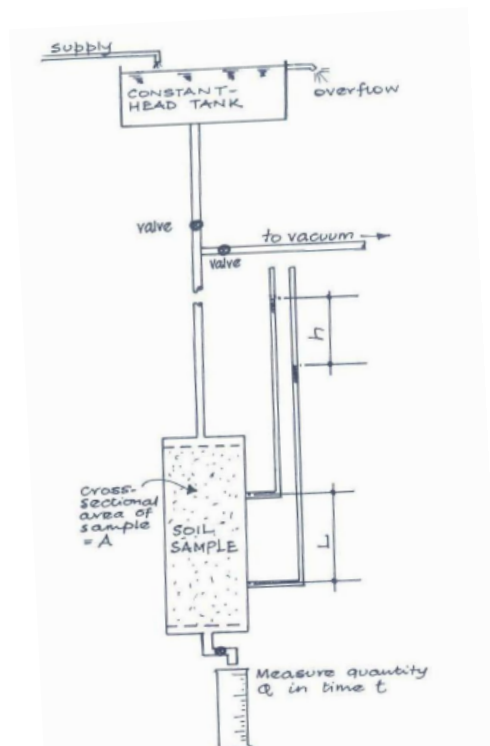


Figure 66. Constant-head permeameter. [B. Vickers. Laboratory work in civil engineering soil mechanics, 1978]

The falling-head permeameter

When the rate of flow through the sample is too small to be accurately measured in the constant head permeameter, the falling head apparatus is used. A coarse filter screen is placed at the upper and lower ends of the sample. The base of the sample is connected to the water reservoir; the top of the sample is connected to a glass standpipe of known cross-sectional area. This pipe is filled with water: as the

water seeps down through the soil sample, observations are taken of time versus height of water in the standpipe above base reservoir level. A series of tests is performed, using different sizes of standpipe, and the average value of the coefficient of permeability is taken.

In this case the permeability is calculated by the next equation:

$$q = -a \cdot \frac{dh}{dt} = A \cdot k \cdot \frac{h}{l} \quad (45)$$

Therefore

$$-\frac{dh}{h} = \frac{A \cdot k}{a \cdot l} \cdot dt \quad (46)$$

Then

$$\frac{A \cdot k}{a \cdot l} \cdot (t_2 - t_1) = -\log_e \frac{h_2}{h_1} = \log_e \frac{h_1}{h_2} \quad (47)$$

and

$$k = \frac{a \cdot l}{A} \cdot \frac{\log_e (h_1 / h_2)}{t} \quad (48)$$

Where

$$t = (t_2 - t_1) \quad (50)$$

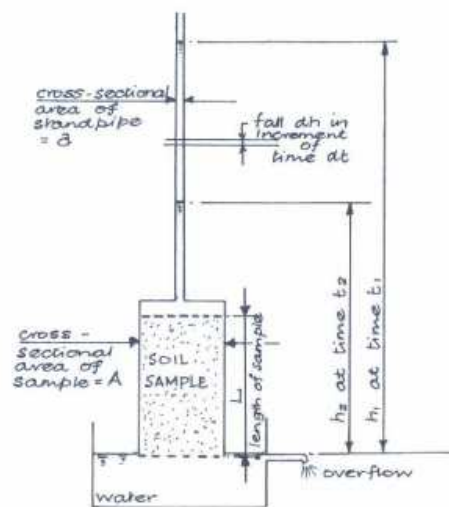


Figure 67. Falling-head permeameter. [B. Vickers. Laboratory work in civil engineering soil mechanics, 1978]

Permeability testing in the triaxial cell

The assessment of soil permeability is, of course, of crucial importance for earth and earth supported hydraulic structures. Given the relationship between confining pressure and pore volume (and hence permeability), it is also crucial that when permeability is measured in the laboratory. The test specimen should be subject to confining pressure. Not surprisingly, therefore, research into laboratory measurement of permeability has concentrated on the triaxial apparatus where confining pressure is routinely applied.

The measurements of permeability can be obtained by means a computer controlled set-up in the triaxial apparatus. This set-up measures permeability by conventional constant head, or by the quicker constant rate of flow method.

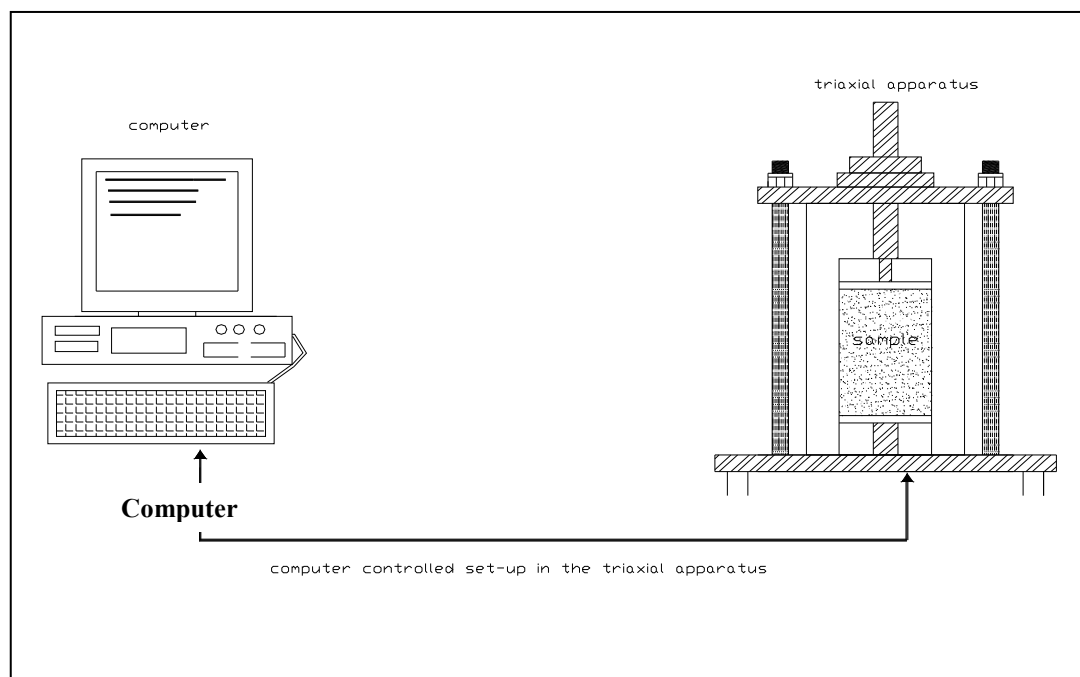


Figure 68. Computer controlled set-up for the measurement of permeability in the triaxial apparatus.

Permeability testing in the oedometer apparatus.

The permeability values for this study have been tested in the oedometer. We describe this method in detail:

The equipment used for the conventional oedometer test can be easily adapted for permeability testing. A cell for that is shown in Figure 69.

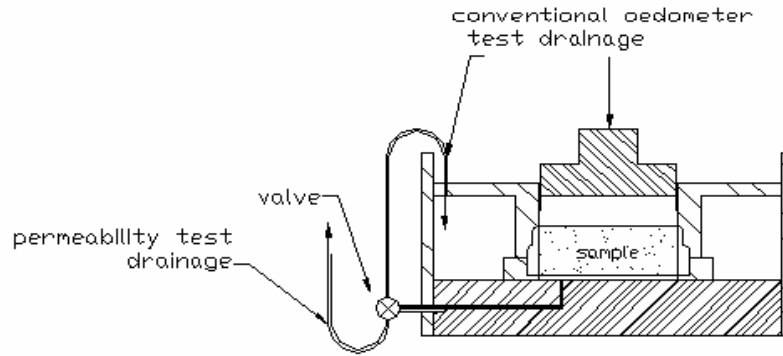


Figure 69. Oedometer cell modified for permeability measurement. [Leroueil et al., 1990]

Drainage from the base of the sample does not occur directly towards the water in the oedometer cell but is controlled by valve which can be used either to connect the top and bottom of the sample for conventional consolidation tests or to connect the base of the sample to a long vertical tube for falling-head permeability test.

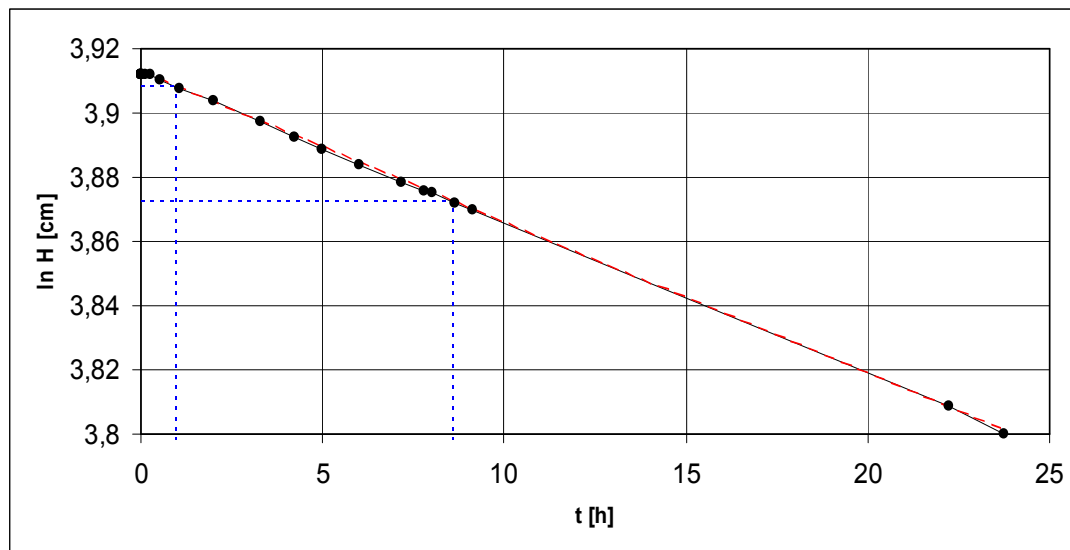
Measurement of permeability can be easily made as part of a programme of conventional oedometer testing: at the end of each 24 h period of consolidation, a permeability test can be carried out over a period also of 24 h. and the sample is then reloaded for a further period of 24 h, and so on.

To calculate the permeability is used the Equation 51.

$$k = 2.30 \cdot \frac{a \cdot L}{A} \cdot \frac{1}{t_2 - t_1} \cdot \log \left(\frac{h_1}{h_2} \right) \quad (51)$$

where a is the cross-sectional of the tube, A is the cross-sectional area of the sample of height L and t_1 and t_2 are the times at which heads h_1 and h_2 are measured in the tube.

The measures read are normally in these times: 6'', 15'', 30'', 1', 2', 4', 8', 15', 30', 1h, 2h, 3h, 4h22h, 23h, 24h. The results are graphic using Excel program. The graphics are like that presented in fig. (2.4):



$$k = 2.30 \cdot a_{byr} L_0 / A \cdot 1 / (t_2 - t_1) \cdot \lg H_1 / H_2 = 1,16E-09 \text{ m/s}$$

Figure 70. Graphic $\ln(H)$ - t to calculate coefficient of permeability by using the equation 51.

Rowe's apparatus method

Rowe's apparatus method tries to solve the most important limitations of constant-head and falling-head apparatus:

- the field stress state cannot be simulated;
- the permeability measurement is limited to the vertical direction.

Rowe's consolidation cell, can be used as a vertical and radial permeameter. The experimental arrangement is showed in Figures 71 and 72. In the vertical flow situation a differential pressure between the base and the top of the sample is

maintained by the inlet and the outlet pressure systems. The flow is indicated by the inlet burette and verified by the outlet burette. A manometer records the pressure difference across the sample. It is seen that a vertical pressure equal to the overburden pressure can be applied to the sample thus simulating the field stress state, and that the radial permeability of relatively large diameter samples can be measured.

Horizontal (radial) permeability measurements can be made using the drainage arrangements shown in Figure 72. A peripheral porous sleeve, 4.5 mm thick is connected to the inlet pressure system and the output flow is collected in a central porous drain connected to the outlet pressure system.

A basic advantage of the Rowe cell is that consolidation and permeability tests can be successively conducted providing data over a range of void ratios. Either inward or outward radial flow is possible.

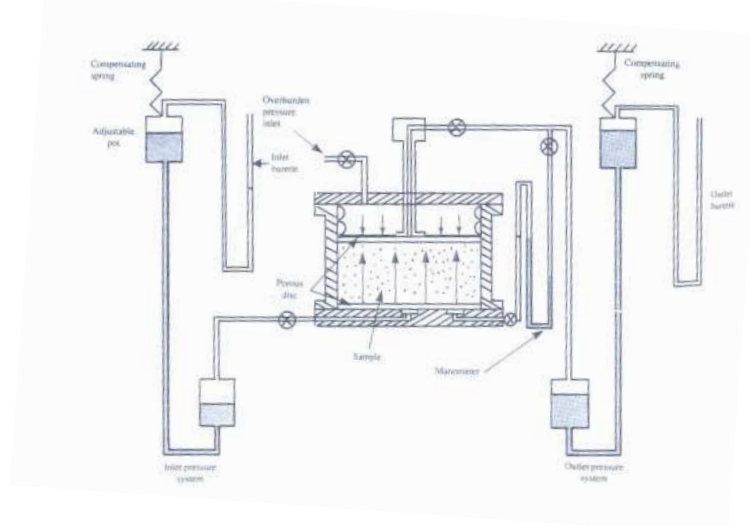


Figure 71. Vertical permeability using Rowe Consolidometer.

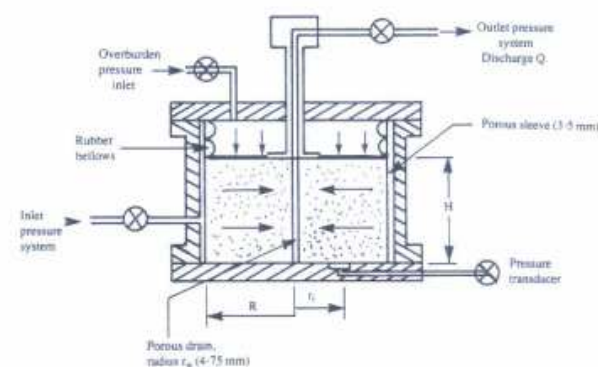


Figure 72. Radial permeability using Rowe Consolidometer.

6.2.2 Precautions required in laboratory tests.

The principal causes of error in laboratory tests are listed below:

- **Air in the sample.** The hydraulic gradient is generally greater in the laboratory sample than in the ground. The resulting pressure drop may cause any air which is dissolved in the soil water to be deposited in the pores. This would considerably reduce the measured permeability. To minimize this effect, all water used for testing is subjected to a vacuum to remove the air from solution.
- **Base exchange.** For the finer grained materials, the nature of the adsorbed ions affects the form of the adsorbed layers, and influences the permeability. In order to prevent exchange of the adsorbed ions, it would be best to test the sample, using the soil water as actually found on site. This is seldom possible in practice, and the water used is usually either distilled or chemically treated to reduce the ion content.
- **Unrepresentative samples.** Since the permeability may vary considerably over quite a small area, the results obtained from a few small samples may not give a very good picture of the average value of permeability over a large area.
- **Anisotropic area.** Many soils have been deposited in approximately horizontal layers of varying permeability. Also, there is often a preferred horizontal orientation of the particles within a layer. As a result, many soils have a much larger permeability horizontally than vertically. The normal form of laboratory tests measures the vertical permeability, which may not have much relevance to cases where the seepage in the ground is predominantly horizontal.

- **Sample disturbance.** In practice, it is almost impossible to obtain undisturbed samples of clean coarse-grained soils. It is therefore usually necessary to carry out laboratory tests on samples which have been recompacted to the in situ density. However, it is impossible to reproduce the stratification and particle arrangement once these have been destroyed, and such tests may not therefore be very relevant to permeabilities in the ground.

6.2.3 Testing permeability in the oedometer apparatus.

6.2.3.1 General requirements.

6.2.3.1.1 Properties of water.

The water used for testing shall not wash out constituents of the specimen, deposit any dissolved or suspended matter in it or alter the colloidal state of the soil. As far as possible, water similar in type to the pore water shall be used, de-aired tap water generally being adequate. Where necessary, the water shall be treated or obtained from a given source so that the natural conditions can be reliably reproduced. [CEN ISO/TS 17892-11:2003 (E)]

6.2.3.1.2 Degree of saturation.

The specimen shall remain saturated during the measurement of the permeability.

Finnish soft clays are fully saturated, but cases in which it is not like that saturation of the specimen can be achieved by applying a back pressure u_0 , as specified in Table 15, which is produced by subjecting the pore water in the specimen to a hydrostatic pressure which shall be maintained throughout the test. But we don't use this system in the laboratory because it is very dangerous to apply these high pressures with traditional equipments, the ones we use.

Table 15. Back pressure as function of initial saturation. [CEN ISO/TS 17892-11:2003 (E)]

Initial saturation Sr(%)	Back pressure u_0 (KN/m ²)
100	0
95	300
90	600
85	900

6.2.3.1.3 Hydraulic gradient.

For testing purposes, the hydraulic gradient may be selected to satisfy practical considerations as long as the flow characteristics given by the gradient complies with Darcy's law. In case of doubt whether the test conditions comply with Darcy's law the hydraulic gradient has to be varied to check it. Where the flow is not linear, the hydraulic gradient in the laboratory shall approximate that in the field.

6.2.3.1.4 Temperature.

Testing shall be carried out at approximately constant ambient temperature ($\pm 2^\circ\text{C}$), with which the temperature of the specimen and water shall be in equilibrium. The temperature shall be measured and recorded.

To obtain reproducible results, the value of k as determined in the tests shall be converted to a reference temperature of 10°C using the following empirical equation, Equation 52, from Poiseuille.

$$k_{10} = \alpha \cdot k_T \quad (52.a)$$

$$\alpha = \frac{1.359}{1 + 0.0337 \cdot T + 0.00022 \cdot T^2} \quad (52.b)$$

where:

T is the water temperature (°C) throughout the test;

k_T is the coefficient of permeability at ambient temperature (m/s)

α is a correlation factor, to be calculated or taken from Table 16. For intermediate values linear interpolation is allowed.

A reference temperature of 10°C equals the average temperature of groundwater. A different temperature may be used where required.

Table 16. Correlation factor α to allow for viscosity of water. [CEN ISO/TS 17892-11:2003 (E)]

Temperature T (°C)	5	10	15	20	25
Correlation factor α	1.158	1.000	0.874	0.771	0.686

6.2.3.1.5 Specimen dimensions.

Specimen diameter and height shall be selected so as to prevent any unhomogenities influencing the test results. The minimum diameter of the specimen shall be 50 mm, and the minimum height 20 mm.

The ratio of maximum particle size to specimen diameter or length shall be not less than 1:5 for non-uniform and 1:10 for uniform soils.

For cohesive (fine grained) soil, the cross-sectional area of the specimen A shall be not less than 1000mm², unless the test equipment requires the use of larger specimens.

6.2.3.1.6 Measurement of standpipe heads.

Standpipes shall have an internal diameter of 3 mm to 4 mm and be located at a minimum of 15 mm from the top and bottom ends of the specimen. In the case of

soils with low permeability, the loss of head between the standpipes and cell is small enough to be ignored so that the difference in head between inlet and outlet may be regarded as being equal to the difference in head across the specimen.

Where the quantities of water passing through the specimen are small, measurement shall be carried out using piezometric tubes or capillary tubes, due consideration being given to the possibility of evaporation falsifying the results.

6.2.3.1.7 Filter blocks permeability.

The filter blocks in the permeameter shall be sufficiently permeable, the permeability being checked at regular intervals, taking the possible influence of valves and connecting tubing into account. The permeability of the filter block shall be at least 10 times bigger than the permeability of the sample.

6.2.3.2 Test execution.

The specimen shall be consolidated by subjecting it to a stress acting vertically and applied via a top plate. In our case we have been testing permeability in the oedometer apparatus, so we test the permeability after the consolidation produced by the oedometer stress. To maintain equilibrium, this stress shall be higher than the additional hydrostatic pressure acting on the specimen in the test. The pore water flowing out of the specimen through the lower filter block shall run off freely.

We can see the apparatus while the execution in Figure 73.



Figure 73. Permeability test by using a burette added to the oedometer apparatus.

A variable hydraulic gradient is produced by a column of water in a standpipe, the level in which falls during the test procedure.

For determining the permeability, the water head in the tube shall be measured at given intervals.

Evaporation, if any, shall be taken into account.

6.2.3.3 Test results.

6.2.3.3.1 Getting permeability from permeability tests.

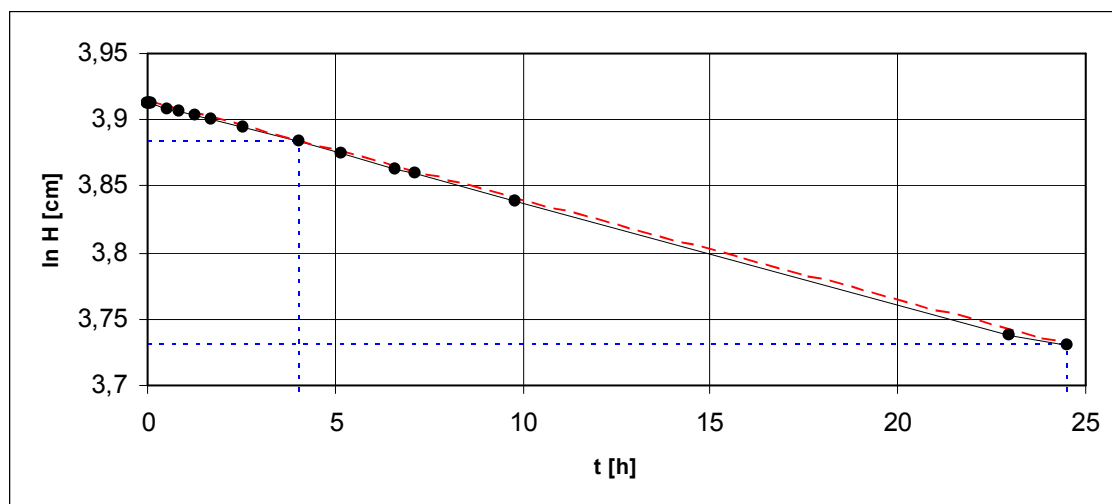
As we told above, we achieve a group of time values and their respective water head in the tube from the permeability test. By using Equation 51 we can get the permeability of the sample. This is what we programmed in Microsoft excel so it provides the k value when we introduce the permeability test data. It can be seen in Figure 74.

Place:	VANTTILA							N:o	4656kss
Piston:	2005-4 (1) TKK 86				Depth:	2,875-2,98			
Oedometer:									
Oed. No.	9k	h ₀ , cm	2,2	A, cm ²	20,0	V, cm ³	43,8	σ ₁ , kPa	100
		L ₀ , cm	1,658	a _{byr} , cm ²	0,5	L ₀ = height of the sample for each load			
Obs.	Stab clay (2% dry mass ratio)								
Sample:									
Soil		Sa-%		Humus, %		ρ _s , t/m ³	2,74	γ ₀ , kN/m ³	12,74
w ₀ , %	110	e ₀	3,38	n ₀ , %	77	s _k , kPa		m _k , g	27,41
S _r , %	87	F		w _L , %		ρ _{d0} , g/cm ³	0,626	γ _{d0} , kN/m ³	6,137211644

Figure 74.a. Initial data.

t(h)	H (cm)
0,001667	50
0,033333	50
0,066667	50
0,116667	50
0,55	49,8
0,833333	49,72
1,3	49,58
1,716667	49,4
2,583333	49,1
4,05	48,58
5,166667	48,16
6,583333	47,6
7,15	47,42
9,816667	46,48
23	42
24,5	41,68

Figure 74.b. Permeability test data.



$t_1 = 4,05$ h $H_1 = 48,6$ cm
 $t_2 = 24,50$ h $H_2 = 41,7$ cm

$$k = 2,30 * a_{lyr} L_0 / A * 1 / (t_2 - t_1) * \lg H_1 / H_2 =$$

8,61E-10 m/s

Figure 74.c. Achieving the k value by using Microsoft excel.

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CODES

- CEN ISO/TS 17892-12:2004 (E)
- CEN ISO/TS 17892-5:2003
- prCEN ISO/TS 17892-6:2003 (E)
- CEN ISO/TS 17892-11:2003

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